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AMERICAN WATER WORKS ASSOCIATION

In this issue:

Spray-away plan of reservoir face (and capacity) lifting at Fontana, Calif.

Journal

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Official reservation forms have been mailed to all members and are returnable to AWWA. Preliminary technical program on p. 48 P&R.

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Mar. 20–22—Montana Section, at Florence Hotel, Missoula. Secretary, Arthur W. Clarkson, Asst. Director, Div. of Environmental Sanitation, State Board of Health, Helena.

Mar. 23–25—Southeastern Section, at Dinkler-Plaza Hotel, Atlanta, Ga. Secretary, N. M. deJarnette, Engr., Div. of Water Pollution Control, State Dept. of Health, 309 State Office Bldg., Atlanta, Ga.

Mar. 26–28—New York Section, at Van Curler Hotel, Schenectady. Secretary, Kimball Blanchard, Riversville Rd., Greenwich, Conn.

Mar. 26–28—Illinois Section, at LaSalle Hotel, Chicago. Secretary, Dewey W. Johnson, Research Engr., Cast Iron Pipe Research Assn., 3440 Prudential Plaza, Chicago.

Apr. 16–18—Nebraska Section, at Cornhusker Hotel, Lincoln. Secretary, Rupert C. Ott Jr., Neptune Meter Co., 2818—21st St., Columbus.

May 15–17—Pacific Northwest Section, at Davenport Hotel, Spokane, Wash. Secretary, Fred D. Jones, Asst. Supt., Water Dept., 306 City Hall, Spokane, Wash.

May 15–17—Arizona Section, at El Conquistador Hotel, Tucson. Secretary, Stanford I. Roth, Supervisor of Water Collections, Div. of Water & Sewers, Phoenix.

Jun. 1–4—Canadian Section, at Royal York Hotel, Toronto, Ont. Secretary, A. E. Berry, Gen. Mgr. & Chief Engr., Ontario Water Resources Commission, Parliament Buildings, Toronto.

Jun. 25–27—Pennsylvania Section, at Hotel Lawrence, Erie. Secretary, L. S. Morgan, Div. Engr., State Dept. of Health, Greensburg.

(*Continued on page 8*)

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(Continued from page 6)

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Sep. 17–19—Ohio Sec., Cleveland.

Sep. 17–19—Wisconsin Sec., Wausau.

Sep. 22–24—Kentucky-Tennessee Sec., Memphis, Tenn.

Sep. 24–26—North Central Sec., Duluth, Minn.

Sep. 28–30—Missouri Sec., Jefferson City.

Sep. 28–Oct. 1—Alabama-Mississippi Sec., Biloxi, Miss.

Oct. 15–17—Iowa Sec., Des Moines.

Oct. 19–22—Florida Sec., North Miami Beach.

Oct. 23–24—West Virginia Sec., Charleston.

Oct. 23–25—New Jersey Sec., Atlantic City.

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Mar. 17–18—Steel Founders' Society of America, Drake Hotel, Chicago, Ill.

Mar. 17–21—4th EJC Nuclear Engineering & Science Conference and International Atomic Exposition, International Amphitheater, Chicago, Ill.

Mar. 31–Apr. 2—Texas Conference on Utilization of Atomic Energy, Texas A&M College, College Station, Tex.

May 5–7—Purdue Univ. Industrial Waste Conference, Purdue Memorial Union Bldg., Lafayette, Ind.

May 12–14—Analysis Instrumentation Div., Instrument Society of America, Shamrock Hilton Hotel, Houston, Tex. For registration information write: H. S. Kindler, ISA Director of Technical Programs, 313—6th Ave., Pittsburgh, Pa.

May 19–23—Short Course on Safety Management Techniques, sponsored by National Safety Council, Chicago, Ill. Write: Director of Industrial Training, National Safety Council, 425 N. Michigan Ave., Chicago 11, Ill.

May 24–31—International Water Supply Congress, Brussels, Belgium (in connection with World's Fair). For information, write: Belgian Organizing Committee, 10 Square Ambiorix, Brussels 4, Belgium.

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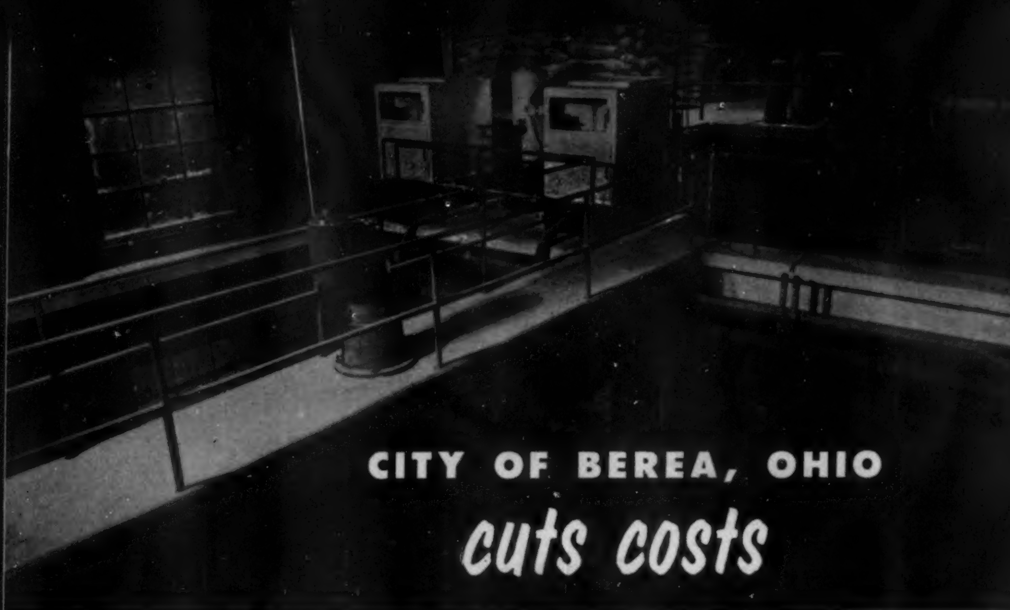
Jun. 10–12—Appalachian Underground Corrosion Short Course, School of Mines, West Virginia Univ., Morgantown, W.Va.

Jun. 22–27—American Society for Testing Materials, Hotel Statler, Boston, Mass.

Sep. 1–13—2nd International Conference on Peaceful Uses of Atomic Energy, Geneva, Switzerland.

Oct. 5–9—Federation of Sewage & Industrial Wastes Assns., Detroit, Mich.

Oct. 13–17—American Society of Civil Engineers, New York, N.Y.



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Value of an Acre-Foot of Water

Edward F. Renshaw

A contribution to the Journal by Edward F. Renshaw, Research Assoc., Dept. of Economics, Univ. of Chicago, Chicago, Ill.

THE people of this nation probably use water in greater abundance than any other people. They demand that water for domestic and municipal use be safe in quality and almost unlimited in quantity, that all arid and semiarid lands be irrigated, that water power be developed in abundance, and that the navigable capacity of navigable water be maintained—all evidence of an enlightened and prosperous people. At the same time they also demand that streams be not polluted, that fish and wildlife habitats be preserved, that scenic features be retained, and that the inalienable rights of individuals, states, and adjacent nations be respected. These uses and demands conflict even in areas of unlimited water supplies. With water limited in many areas and with demands thereon increasing, competition is bound to develop. Moreover, people are prone to look into the future and wonder if decisions and dedications made today concerning the use of water will prove sound tomorrow; consequently, they are interested not only in today's conflicts, but they also endeavor to minimize those of tomorrow (*1*, Vol. 3, p. 1097).

The US has reached a point in the development of its economy when only rarely can use be made of water for some particular purpose without adversely affecting its use for some other purpose. For instance, diversion of water for irrigation can result in lower navigable depths, lower power potentials, limited ability of streams to dilute and carry away industrial and human wastes, decreased recreational values, destruction of fish and wildlife habitats. It may permit the intrusion of salt water into bays and estuaries, with a consequent modification of the chemical composition of coastal waters, their character, and their aquatic life. The water that does return to streams after irrigation use may contain salts which limit other beneficial uses. The construction of power reservoirs, stock tanks, ponds, and other water-holding devices increases evaporation and thus

may have the same effect as actual withdrawal for domestic, industrial, and irrigable use. Dams can seriously interfere with the migration of fish, destroy spawning grounds, and inundate scenic, recreational, and agricultural lands, thereby reducing their values. Variations in the discharge of water through power turbines and the discharge of water with a low oxygen content can greatly reduce the capacity of a stream to decompose organic wastes and, hence, its ability to act as an agent of waste purification. Overloading a stream with waste elements can destroy fish, impose costs on downstream users, and may seriously endanger the public health. The list is illustrative and by no means exhaustive. Conflicts in the use of water have already received widespread attention (*1*, Vol. 3, pp. 1095–1106), and can reasonably be expected to become of increasing importance as time goes on.

The question remains, how shall conflicts in water usage be resolved?

Resolution of Conflicts

It has been suggested in numerous water policy documents that conflicts should be resolved so that water is put to its highest economic use (*2*, Vol. 1, p. 55), its optimum use (*1*, Vol. 1, p. 31), or its most productive use (*3*). In general, what is meant by the terms economic, optimum, and productive is never clearly spelled out. The implication, however, seems to be that where conflicts exist, water should be put to uses which can be expected to yield the highest real returns, monetary and nonmonetary. From an economic point of view, therefore, best usage is related to the willingness of people, either individually or collectively, to pay for the use of water for alternative purposes. If conflicts are to be resolved, value comparisons must be made.

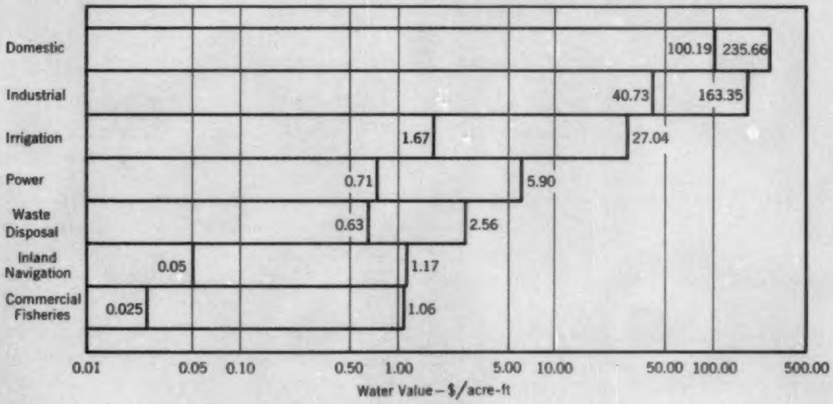

Fig. 1. Values of Water

Methods used for determining the various values per acre-foot of water are explained in detail in the text. Bars show maximum and mean values, for purposes of comparison.

While the inference seems clear that value comparisons should be made within and between water use categories in order to aid the various agencies of government in their endeavor to resolve conflicts, comprehensive value comparisons have not been made; and, as a result, little is known as to how society should be prepared to shift from low to higher water usages as the forces of economic and population growth press against an inadequate total supply. Adequate supply would imply that water is sufficiently plentiful as to be a free good that need not be rationed among users. The President's Materials Policy Commission has estimated, for instance, that between 1950 and 1975, municipal and industrial water requirements will increase 50 and 170 per cent, respectively (*2*, Vol. 1, p. 51). It was further suggested by this group that, as time goes on, a shift, particularly in the western part of the US, from the agricultural use of water for irrigation to industrial use will be increasingly warranted in terms of relative economic values (*2*, Vol. 5, p. 86).

Use Categories

In this article an attempt is made to summarize information on values of water in such a way that comparisons can be made within and between seven water use categories. The data, which are summarized in Fig. 1, are intended to be illustrative of possibilities for making comparisons rather than in any sense a final word as to the value of water in various uses. Their purpose is to give an idea of the relative values that are involved in water usage.

The basic data used in determining the values of domestic and industrial water in Fig. 1 are compiled from a 1950 survey of 416 cities with populations of 10,000 and over (*4*). It is assumed that domestic users are included in the monthly rate bracket for 1,000 cu ft of water. The maximum value per acre-foot was determined by multiplying the maximum rate for 1,000 cu ft of water by the acreage factor 43.5; the mean value, by multiplying the mean rate for 1,000 cu ft of water (380 cities reporting) by the acreage factor.

For industrial use, the assumption made is that industry uses a larger quantity of water and thus is best represented by the monthly rate bracket for 1,000,000 cu ft of water. Again, the maximum and mean rates for 1,000,000 cu ft of water were multiplied by the acreage factor of 0.0435 to obtain the respective values per acre-foot. It should be noted, however, that large industries as a rule find it less costly to supply their own water. These estimates, while they represent the best statistics available, are not unbiased in the sense that they are representative of all industrial users. Inclusion of all users would tend to lower the mean rate for 1,000,000 cu ft of water consumed.

Irrigation data are based on 1949 figures: the maximum cost of water to farms per acre foot in San Diego County, Calif., and the mean county cost of water to farms per acre-foot in 343 counties from 21 states (*5*).

In order to arrive at a value of water used to generate hydroelectric power, an output of 1 kwhr/acre-ft of water for every foot of head is assigned; this tends to be a maximum output and is approached only in the most modern and efficient generating plants. It is further assumed that 1 kwhr of electricity is worth 1 cent; this value would be too high without the further as-

sumption that the energy produced is used for peaking purposes. Frank Weaver of the Federal Power Commission has pointed out that hydro power costing 10–15 mills per kilowatt-hour may be less costly than steam power used in the peak of the load, even though power from steam plants might cost 6–8 mills per kilowatt-hour at system load factors (*6*). Based on these simplifying assumptions, the value of an acre-foot of water is a direct function of the generating head. The maximum is the assumed value of an acre-foot of water at Hoover Dam, where the maximum generating head is 590 feet. The mean is an assumed value of an acre-foot of water, where the generating head is 71 ft; this head is obtained by averaging the structural height of 223 dams constructed by the Bureau of Reclamation as of 1954 (*7*) and by reducing the average structural height by 32 per cent, the average percentage difference between structural height and maximum head for eighteen Bureau of Reclamation dams (*8*). It should be noted that, although it is possible to obtain a sizeable and fairly representative distribution of dam heights from reclamation data, only a few of the dams constructed by the bureau are power dams.

The value of water as an agent of waste purification is essentially a function of the alternative cost of waste disposal and treatment and the extent to which society will permit the receiving water to become polluted. The data which exist with respect to the alternative cost of waste disposal and waste treatment are either incomplete or lacking in comparability; for this reason, it is impossible to determine with a high degree of accuracy the actual cost distributions. For purposes of illustration, however, it will be assumed that the alternative cost of waste disposal is 1 cent per day per population equivalent (it would not be difficult to find places where the cost is much higher). Under this assumption, the value of water as a waste disposal agent is a direct function of dilution. The maximum rate is based on the value of an acre-foot of diluting water flowing at the rate of 2 cfs per 1,000 population. Babbitt has noted (*9*) that, between a minimum limit of 2 cfs and a maximum limit of 8 cfs of diluting water per 1,000 population, the success of dilution is uncertain with respect to the prevention of a nuisance. The mean is figured as the value of an acre-foot of diluting water flowing at the rate of 8 cfs per 1,000 population.

The value of water as an artery for commerce can be obtained by making suitable adjustments in the Corps of Engineers' estimates of traffic savings on various waterways in 1953, multiplying the adjusted savings by the 1951 traffic and by dividing this product by the average flow at some point along the waterway. The Corps' 1953 estimate of traffic savings is first adjusted for a 16.5 per cent rise in cost of transportation as measured between 1950 and 1953 by the consumer price index and then reduced by 50 per cent to allow for a straight-line demand for navigation services (*10*).

The maximum navigational value of an acre-foot of river water was computed in terms of the average discharge at Bellaire, Ohio, on the Ohio River, for the 11 years of record, 1942–52. This assumes an average flow of 41,990 cfs, a traffic of 56,570,000 tons (the largest tonnage recorded for an individual waterway in 1951), and a traffic saving of \$0.63 per ton. If the average

flow at Metropolis, Ill. (254,000 cfs, 1921–45) which is located nearer the mouth of the Ohio River, is used instead of the flow at Bellaire, the value of navigational water falls from $1.17 per acre-foot to $0.19; the value of navigational water on the Ohio is critically dependent on the point at which stream discharge is measured. The mean value was computed from the navigational value of an acre-foot of Cumberland River water in terms of the average discharge at Smithville, Ky., 1921–45. Of the nineteen most important waterways in 1953, the Cumberland ranked tenth in terms of total tonnage transported. This assumed an average flow of 27,800 cfs, a traffic volume of 1,780,000 tons, and a saving per ton of $0.62.

The value of an acre-foot of fishery water is conceived to be the commercial value of the yearly catch. While the theoretical value of the fish that could be obtained from an intensified pond culture overshadows even the highest price paid for irrigation water, the fact that pond culture is of little commercial importance in the United States raises doubts as to the relevance of theoretical values. In general, the value of inland fisheries per acre-foot of water is probably very low. The maximum rate assumes a production of 1 lb of fish per day per acre-foot of water over a 150-day growing season. A growth rate approaching this order of magnitude has been noted in relation to Minnesota sucker-rearing ponds by John Dobie (*11*). A dollar value for the fish produced is obtained by assigning the average price per pound ($0.0708) received by United States and Alaskan fishermen in 1950.

The mean is obtained by dividing into the value of the commercial fish catch (*12*) from Lake Michigan in 1950 ($3,661,209) the area of Lake Michigan (14,295,040 acres) times a depth factor of 10 ft. If the commercial value of the 1950 catch were divided by the true volume of Lake Michigan, the return per acre-foot of water would be quite small indeed, since the maximum depth is estimated to be 870 feet. Smith and Chapin (*13*) have noted, however, that plankton, the most important vegetation of the sea, can only live in depths down to about 250 ft. It is to be expected, therefore, that a large fraction of the total volume of Lake Michigan does not contribute much to total fish production.

Considerations of Value

In each category value is estimated at a point or within an area of consumptive use. It should be noted that the same water may be used for several simultaneous purposes, and hence the aggregate value of an acre-foot of water at any point of use is a sum of individual use values. Furthermore, to the extent that the water is returned to a stream or a natural reservoir, the same water may be used to generate values at other points of use.

It must be emphasized that in each category the value of water as depicted in Fig. 1 is a gross value; it could not be collected as a net rent from water users, since other costs are incurred and included in the price people pay for the use of water. For municipal and industrial water, there are the costs of purification and distribution; for irrigation, the costs of diversion and distribution; for power, the costs of generating and building a generating head; for waste disposal, there are the social costs of polluted water which

are usually borne by other water users; for navigation, the costs of constructing locks and dredging channels; and for commercial fisheries, there are the costs of maintaining the fish population and harvesting the commercial crop. In large measure, the value estimates in Fig. 1 reflect these other costs and do not represent the net value of water as a consumption good or a factor of production. Not knowing exactly what the net value of water is, since in most instances there does not exist an active market in water rights alone, the best estimate of relative water values may well be either distributions of market prices paid for water at points of use or distributions of actual costs incurred in putting water to work. It should be borne in mind that a comparison of gross water values is often inaccurate, since an arbitrary decision to include or exclude a given cost element could affect relative values.

An analysis of Fig. 1 reveals that of all the uses for water, domestic use is by far the most valuable; industrial usage is relatively the next most valuable use. A very large gap exists between the value of domestic and industrial water and the value of water for the purposes of irrigation, power, and waste disposal. An extensive overlapping of relative values exists with respect to the latter three usages. In general, the values generated by inland navigation and commercial fisheries are least important.

In constructing Fig. 1, no attempt was made to assign a monetary value to recreational and aesthetic attributes of water. For some bodies of water, these attributes could conceivably overshadow other usage values. Furthermore, no attempt was made to isolate the true value of water in the demand sense, where the quantity of water used for a particular purpose is a function of relative price, income, and other variables. Values in the demand sense can be thought of as the maximum amount people would be willing to pay for the use of water in any amount or direction rather than forego the amount or use entirely.

Conclusion

Some study has been made (*14*), and others are in the process of being made on various types of demands for water. The estimates in this article are presented in the hope of stimulating thought and further work in the area of making value comparisons—an area which should not go undeveloped if conflicts in water usage are to be resolved in terms of what is optimal from the standpoint of the nation as a whole.

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Correction

The committee report "Trends in Air-Conditioning Use and Regulation" (January 1958 Journal, Vol. 50, pp. 75–96) contained several editorial errors. On p. 77, in the tabular portion showing air conditioning installed in some major cities, symbols designating footnotes should be an asterisk (*) and a dagger (†)—in that order—rather than the dagger and double dagger (‡) shown. The reference to "Table 12" in col. 1, p. 81, should read "Table 9." In Table 5 on p. 82, in col. 1, line 21, the first parenthesis should precede the word "nonconserved" rather than "use," to give "5 ton or over (nonconserved use in 1½ ton or less prohibited)." In the italic portion of Fig. 2 on p. 83, the phrase "tonnages shown or greater" should read "tonnages greater than figures shown." In Table 7 on p. 84, the last entry in col. 2 should be "11," not "9"—as shown—and, similarly, the last entry in the "Per Cent of Total" column should read "18.7," not "15.3." In the same table, the footnote symbol (§) should follow the entry "Miscellaneous" at the bottom of the first column.

Irrigation Use of Water

Luther B. Bohanan

A paper presented on Nov. 1, 1957, at the Chesapeake Section Meeting, Washington, D.C., by Luther B. Bohanan, Agricultural Economist, formerly with Washington Dist., US Army Corps of Engrs., Washington, D.C.

WITH the development of lightweight, fast-coupling sprinkling equipment, the practice of irrigation has become relatively widespread in the humid agricultural area. Thus, as more areas are brought under irrigation, the subject of irrigation water use is becoming increasingly more important. In light of these developments, this article discusses some of the critical aspects of irrigation water use.

Broadly speaking, the problems associated with irrigation water use can be grouped under two headings, technological problems and institutional problems. The technological problems are those associated with or have their origin in natural-science data, while the institutional problems are those that have their origin or are associated with social-science data.

There are many technological problems associated with irrigation water use, among them being water requirements for optimum plant growth, moisture deficiencies, available soil moisture, design and operation of the irrigation systems, timing and frequency of irrigation, depth of irrigation application, amount and manner of fertilizer application, and pest control. This article, however, will be limited to a consideration of the determination of water requirements for irrigation.

In planning for a supply of water to meet the total demands for water for irrigation, it makes little difference whether the planner is considering a water supply for an individual farm or for some larger areas, as the factors to be considered will be the same in both instances. The planning for an individual farm, however, may be done with considerably more accuracy. In this article, attention will be centered more toward the planning of a water supply on a watershed basis rather than for an individual farm.

Optimum Plant Growth

In planning for a water supply for irrigation, it is necessary to have a knowledge of the water requirement for optimum plant growth. Plant and soil scientists have been studying this problem for many years. Although no exact method has been developed, a number of methods have been devised which give reasonably satisfactory results. Among the methods developed so far, the evapotranspiration, or consumptive use, method has had the most widespread use or application. By this method, it is possible to estimate the amount of water which will be lost from the soil by direct evaporation and by transpiration of plants through the process of photosynthesis.

Evapotranspiration is determined by many factors such as temperature, humidity, windspeed, precipitation, and radiation. From these data, it is pos-

sible to compute daily evapotranspiration rates, but this is a time-consuming process and very few weather stations record all these data. A relatively simple method for calculating a monthly evapotranspiration or water requirement for plants has been developed by Blaney and Criddle (*1*). Disregarding all other factors, according to this method, consumptive use or evapotranspiration varies with the temperature and daytime hours. Their formula for obtaining a monthly evapotranspiration rate is $U = KPT/100$, where U is the monthly consumptive use in inches (for evaporation and transpiration); K, the crop use coefficient; P, the monthly fraction of daytime hours of the year; and T, the mean monthly temperature (Fahrenheit).

TABLE 1

*Evapotranspiration Rates, May–September, for Westernport, Md.**

Month	T	P	$T \times P$	K	$U = KPT$	
					Monthly *in.*	Daily *in.*
May	62.1	0.0997	6.19	0.8	4.96	0.16
Jun.	69.6	0.1002	6.97	0.8	5.70	0.19
Jul.	73.5	0.1016	7.47	0.8	5.89	0.19
Aug.	72.0	0.0951	6.85	0.8	5.58	0.18
Sep.	66.7	0.0838	5.59	0.8	4.50	0.15

* See text for explanation of symbols.

By using this formula, an evapotranspiration rate can be calculated for any area. Using Westernport, Md., as an example, Table 1 shows how the formula can be applied. When the monthly requirements are added, this gives a total of 26.46 in. of water which will be needed in the Westernport, Md., area to meet moisture requirements for optimum plant growth for the growing season from May 1 to September 30.

With the evapotranspiration rates, it is possible to compute a daily soil moisture balance and, thereby, determine the amount of water needed to supplement rainfall to maintain optimum growth for any period of time. The computation of such water requirements is predicated on the assumption that there is a basic amount of soil moisture available in the root zone at the beginning of the growing season. It is generally assumed that the base amounts of available soil moisture in the root zone range from 1 to 5 in., depending upon the type of soil.

The computation of a moisture balance is based on the principle that the moisture content of the root zone at a given time equals that at a previous time plus the difference between income (precipitation) and outgo (evapotranspiration) over the period elapsed. Thus, by successively adding daily precipitation to and subtracting daily losses (evapotranspiration) from the base amount, the moisture balance can be estimated successively for each day for as much as a month or a complete growing season.

Total Water Requirements

Using the method outlined above, moisture deficiency from the viewpoint of planning a water supply for irrigation can be predicated on a probability analysis of the frequency of drought occurrence or on the basis of maximum drought of record.

Using the maximum drought of record as the design criterion, it is possible to estimate total water requirements for irrigation. Besides the criterion of maximum drought of record, other assumptions have to be made. First, the crops to be irrigated will be corn, hay, pasture, and orchards; and all crops will be irrigated to depth of

TABLE 2

Water Required for Irrigation, Based on Maximum Drought of Record, Westernport, Md.

Month	Water Requirement *in.*	Moisture Deficiency (1930) *in.*	Irrigation Water Required*	
			in.	*acre-ft*
May	4.96	0.22	0.31	0.03
June	5.70	2.35	3.36	0.27
July	5.89	3.87	5.53	0.46
August	5.50	3.33	4.76	0.40
September	4.50	3.44	4.91	0.41
Total	26.63	13.21	18.87	1.57

* Based on 70 per cent efficiency of the irrigation system.

24 in. Secondly, there will be 3 in. of available soil moisture to a depth of 24 in. at the beginning of the growing season. Third, the growing season will be from May 1 through September 3.

Using these criteria, the total water requirements for the Westernport, Md., area are set forth in Table 2. Taking into consideration that an irrigation system will operate at 70 per cent efficiency, irrigation requirements to make up deficiencies are 1.57 acre-ft for the growing season of May 1–September 30. This is based on the moisture deficiencies of the 1930 drought.

Once water requirements per acre-foot have been determined, it is a relatively simple matter to get gross water requirements for the total acres to be irrigated.

There are still a number of technological problems associated with irrigation, including the correct time of the application and the improvement of efficiency of the system. As time passes, it is expected that these problems will be solved, thereby making possible a greater expansion of irrigation water use.

Increase in Irrigation

If realistic plans are to be made to provide water for irrigation, attention will have to be given to the growth of the number of acres under irrigation.

As with a great many phenomena, the available data for estimating the growth of irrigation are rather meager, particularly for humid areas. The US Bureau of the Census has been collecting data on various facets of irrigation since 1890 for the western states, but has obtained data for all the humid states only since 1935 on a state basis. For data on a county basis (which could be converted to a small watershed basis), the Bureau of Census has obtained irrigation data for only the 1950 and 1954 censuses of agriculture. Thus, estimating the growth of irrigation from such meager data is a rather hazardous operation. This discussion of the growth of irrigation will be confined to 28 of the humid states, excluding Arkansas, Florida, and Louisiana.

In the period 1949–54, the land under irrigation in the 28 humid states increased by 435,292 acres, while in the entire United States it increased by 3,764,700 acres (Table 3).

As both cropland and pasture land are irrigated, the source of future acres to be irrigated would come from these categories of land use. Since there is great difficulty in getting comparability of data for all areas, due to differences in the agriculture of the humid states

TABLE 3

*Land Under Irrigation**

Area	Irrigated Land—*acres*		Increase *acres*
	1949	1954	
28 humid states	152,586	587,878	435,292
United States	25,789,453	29,552,155	3,764,700

* Source: Census of Agriculture, 1954.

as contrasted to the arid states, it seems best to study the growth of irrigation as it relates to cropland harvested and cropland used only for pasture.

The method for estimating the number of acres under irrigation was developed empirically. When the data for total acres irrigated in the United States for the period 1900–54 were plotted on log-log paper, the results gave essentially a straight line. This indicates that irrigation in the entire United States for this period increased at an increasing rate. As for the 28 humid states, it is assumed that irrigation will develop in a similar manner to that for the United States—that is, irrigation in these states would increase at an increasing rate.

The basic data for which an average annual rate of growth was determined are given in Table 4. From these data, it was found that, during the period 1949–54, the number of acres under irrigation in the 28 humid states increased at an average annual rate of 0.04 per cent of the total specified cropland, while in the United States, the rate of increase was 0.18 per cent.

Since the assumption is that irrigation will increase at an increasing rate, a further assumption is added to the effect that the incremental change in the rate of increase for the 28 humid states will be such as to be equivalent to the rate of increase in the entire United States. This gives an incremental change in the rate of increase of 0.003 per cent.

TABLE 4

Rate of Increase in Irrigation, 1949–54

Area	Increase in Land Irrigated *acres*	Specified Cropland* *acres*	Rate of Increase	
			Total *per cent*	Avg *per cent*
28 humid states	435,292	208,039,591	0.2092	0.0418
United States	3,764,700	399,808,371	0.9416	0.1882

* Harvested land and land used only for pasture.

TABLE 5

Estimated Land Under Irrigation in 28 Humid States, 1960–2000

Year	Acres Irrigated
1960	1,059,410
1970	2,442,472
1980	5,762,415
1990	7,175,774
2000	10,962,095

With the basic data of an average annual rate of increase of 0.04 per cent and an incremental change in the rate of increase of 0.003 per cent, the number of acres under irrigation was estimated by use of the following formula:

$$N = Y\ (a + \tfrac{1}{2}y)\ n + n_o$$

where N is the number of acres under irrigation; Y, the number of years from 1954; a, the annual rate of increase as of 1954; y, the incremental change in rate of increase from 1954; n, the number of acres of cropland harvested and cropland used only for pasture in 1954; and n_o, the number of acres irrigated in 1954.

Using the above rates of growth in the formula and the amount of cropland harvested and used only for pasture in 1954 as the starting point, the number of acres under irrigation at future specified dates is given in Table 5. According to these estimates, there would be approximately 11,000,000 acres under irrigation in these states by 2000.

Inasmuch as irrigation is a completely consumptive use of water, it can be seen readily that tremendous amounts of water will be needed to meet future irrigation requirements if these projected growth rates materialize.

Institutional Problems

As used in this article, an institutional problem is one that arises from the inefficient functioning of a behavior pattern. The behavior patterns referred to in this article are the habits, customs, and laws that relate to the use of water. The doctrine of riparian rights is the best example of a behavior pattern affecting water use. Simply stated, the doctrine of riparian rights means that every landowner who owns land adjacent to a natural stream or body of water is entitled to the flow of that body of water, undiminished in quantity and undiluted in quality. This places a considerable legal hazard on a person who wants to make a consumptive use of water.

What is needed is a method or system that will encourage beneficial use of water and protection for individuals who are making such a beneficial use. This will require considerable overhauling of laws and customs.

If everybody's needs were to be met, it would require the establishment of priorities and permits. Furthermore, it would require more than a piecemeal approach to water resource use and development. If water requirements were to be met for an expanding population for domestic household purposes, commercial and industrial purposes, recreation, and irrigation, that would call for the development of a comprehensive approach. The best answer seems to be in the direction of a river basin approach. Although planning on a small watershed basis may be sufficient to solve local problems, it does not solve the problem for those further downstream.

If water requirements were to be met for all manifold purposes for which there is need, this would mean a multipurpose approach to water resource development—in other words, a program for the development, conservation, and use of every drop of water that enters any given area.

Reference

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Irrigation in the Midwest

Excerpt From 'Business Conditions'

A condensation of an article which appeared in the December 1956 issue of Business Conditions, *published by the Federal Reserve Bank of Chicago, Chicago, Ill.*

ARTIFICIAL watering of growing crops traces back through antiquity to the transition of mankind from pastoral societies to agrarian cultures. The ancient pyramids of Egypt, for example, were erected by workers fed through the use of a crude type of irrigation.

A farm equipment trade publication has estimated that by 1960 a total of 400,000 farms in the United States will be irrigating 40,000,000 acres, about 10 per cent of the cropland. In 1954, some 300,000 farms irrigated less than 30,000,000 acres. This projected increase would require the investment of something like $500,000,000 in new capital, most of it for sprinkler irrigation equipment. Although the above projection may be overly optimistic, it nevertheless indicates the potentialities that this development may contain.

Although east of the Great Plains average annual precipitation is sufficient for most temperate zone crops, the moisture supply varies considerably from year to year, as does its distribution within the year. Hence, even in humid areas natural precipitation does not guarantee a fully adequate moisture supply throughout a crop's entire growing season. This can cut the yield substantially, especially if the water deficiency occurs at critical stages in the plant's development. Consequently, the output of many crops can be raised considerably by irrigation, even in humid areas.

Artificial watering on a commercial scale is a rather expensive practice. Installation of the equipment requires a large investment outlay, and annual costs of operating the equipment are high. For this reason, in the eastern half of the United States—where irrigation is not subsidized by government-developed water supplies—the usage gained its first footholds in areas specializing in the commercial production of vegetables, because these return a high value of output per acre and are especially sensitive to a steady supply of moisture.

In this part of the country, the water is typically applied by sprinklers—large-size relatives of the type commonly seen watering lawns. The water is obtained from wells, streams, or ponds, and is pumped to the sprinklers through portable pipes. Where a considerable area is involved, the apparatus is moved around the field in order to supply moisture to all parts of it. Obviously, the development of lightweight aluminum pipe has been a boon to this activity.

Increase in Irrigation

In 1954, according to the Bureau of the Census, a total of 2,600,000 acres of farmland was irrigated in the 31 states east of the Great Plains, that is,

east of a line from North Dakota to Texas. Although ten times as many acres were irrigated in the West, the total in the humid region represented a jump of 73 per cent during the previous 5 years, compared with an increase of only 10 per cent in the West. In the humid area, very large percentage gains were scored by a number of

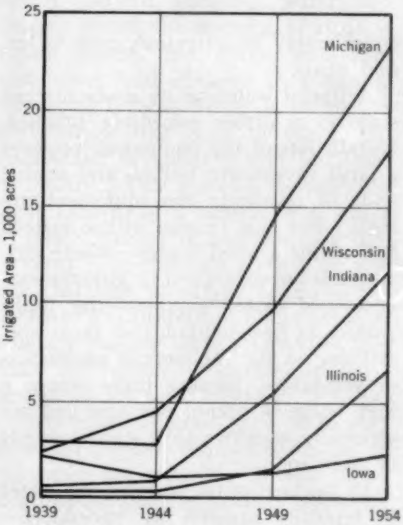

Fig. 1. Irrigated Acreage in District States

Land under irrigation in the area covered by the Seventh Federal Reserve District has expanded rapidly since 1939 in all states except Iowa.

states along the Atlantic seaboard and in the Appalachian and Delta regions, as well as in the corn belt states of Missouri, Ohio, and Illinois.

Irrigated acreage in Illinois leaped from 1,510 acres in 1949 to 6,789 in 1954. (Total cropland harvested in Illinois comes to about 20 million acres.) Several other states in the Seventh Federal Reserve District * had earlier experience with the practice. Michigan and Wisconsin had significant amounts of irrigated acreage already in 1949 (Fig. 1). Nevertheless, those acreages nearly doubled in the succeeding 5 years, with most of the increase occurring on fruit and vegetable farms, the types that previously had accounted for most of the irrigation in that region. In Illinois, Indiana, and Iowa most of the gain in sprinkled acreage occurred on cash grain (primarily corn) farms, although Indiana also showed a significant addition on grain and meat animal farms.

Irrigation in the Seventh District is not primarily a small farm phenomenon. In both 1949 and 1954 the bulk of the sprinkled acreage was found on farms exceeding 219 acres in size (Table 1). The largest percentage increase between those two dates, however, occurred on farms in the 99–219-acre category, which includes the majority of district farms.

Irrigation of Corn

A considerable amount of experience has now accumulated concerning the irrigation of corn in the Midwest. Purdue University specialists report that irrigation doubles and triples corn yields in some sections of Indiana. A 3-year test in northern Illinois showed an average corn yield of 144 bu per acre on sprinkled land against a 50-bu yield on nonirrigated soil. In a 4-year Wisconsin test, nonirrigated corn averaged 45 bu per acre whereas full irrigation plus heavy fertilization boosted the yield to 96 bu. Tests by Iowa State College on heavy Iowa soils in 1955

* The Seventh District covers all of Iowa, major portions of Illinois, Indiana, and Michigan, and approximately one-half of Wisconsin.

showed an average gain of 40 bu per acre due to irrigation.

These tests were conducted for periods of only 1–4 years, whereas a longer span of time would be desirable for conclusive results. An Iowa study of rainfall records has disclosed, however, that, on the average, there were more than three dry periods in June, July, and August each year during the past 20 years at Ames. A dry period was defined as ten or more consecutive days during which there was no more than ¼ in. of precipitation on any day. "Every year in the past 20 had at least one period of 13 days or more with no effective rainfall; sometimes these periods last 20 days or longer."

TABLE 1

Irrigated Acreage in Seventh Federal Reserve District

Size of Farm *Acres*	Acres Irrigated		Total Crop Acres, 1954
	1949	1954	
100 or less	7,067	11,251	7,438,963
100–219	5,065	12,497	30,036,641
Over 219	20,531	38,847	34,360,277
Total	32,663	62,595	71,835,881

Source: US Bureau of the Census.

This state study suggests that irrigation of corn might be warranted merely from the standpoint of supplying supplemental moisture during dry spells. An important additional advantage of artificial watering, however, lies in the fact that it permits other changes in crop production practices. For example, it has been known that highest corn yields can be obtained from good soils if heavy applications of fertilizer are used along with dense stands of plants—that is, more than 15,000 per acre. Moisture deficiency can sharply reduce the effectiveness of fertilizer, however, and dense stands may actually reduce yields in dry years. The sheer threat of inadequate water—that is, the mere uncertainty connected with lack of irrigation—undoubtedly has held planting and fertility practices below optimum levels. Several observers report that installation of irrigation equipment usually should be accompanied by a 50–100 per cent boost in fertilizer application in order to utilize soil potentialities most fully. "For best results, fertility, water, and plants per acre should all be high."

Costs

In view of the results obtained through irrigation, it may seem surprising that the practice is not already more widespread in the Midwest. Undoubtedly, part of the explanation is provided by the cost of the necessary equipment. Pump, pipe, and sprinklers cost from $40–$80 per acre irrigated, depending on the location of the water supply, the layout of the fields, and the farmer's taste in machinery.

The cost of obtaining the water supply is even more variable. Irrigation requires a lot of water; almost 30,000 gal are needed to cover 1 acre with 1 in. of water. Sprinkler systems in this region vary considerably in capacity, squirting from 200 gpm to 1,500 gpm. At the rate of 500 gpm it takes about 1 hr actual sprinkling time to cover an acre with 1 in. of water.

Sometimes the water is secured from streams that flow throughout the year, and sometimes small reservoirs are constructed to impound surface runoff in rainy seasons. Frequently, shallow wells or sump holes are dug near creeks to utilize both surface runoff and underground water. Where the flow from these holes is small, the

water is pumped into reservoirs over a period of time, with the supply being drawn down when irrigation is necessary. Other systems rely exclusively on underground water, although some of them also utilize small reservoirs for storage.

Most of the Seventh District is underlaid by water-bearing strata yielding more than 50 gpm to individual wells of suitable depth and dimensions. Moreover, the ground water level is close to the surface in the Midwest compared with the West, where deep wells are also used to some extent to provide irrigation water. Some Midwest farms have underground supplies capable of yielding more than 800 gpm from a single well. It is the judgment of authorities that irrigation as currently practiced in the Midwest is feasible for a high percentage of farms in this region.

If the required water can be secured from a stream, pond, or shallow well, the cost of securing the supply may be relatively low. The cost, however, mounts rapidly if a deep well with a wide bore, sometimes up to 3 ft in diameter and several hundred feet deep, is required. A deep well may cost $5,000 or more. It behooves a farmer to seek technical advice before having such a hole drilled; its particular location can make a lot of difference in the cost. Also, the water laws should be investigated before a large investment is made in an irrigation system. Although these laws are quite indefinite in most Midwest states, in general they prohibit one user from interfering with normal uses of other users.

Complete sprinkler systems—water supply plus distributing equipment—require an investment of from $40 to $200 per acre, with the average around $90 in the Midwest, according to one company which fabricates and installs the equipment.

Various authorities are pretty well agreed on the annual cost of operating a sprinkler system. Iowa State College places the figure between $20 and $30 per acre. A company that sells the apparatus estimates average annual cost per acre for a typical installation in this region as follows:

Item	Annual Cost per Acre
Interest	$ 4.50
Insurance and taxes	0.45
Depreciation	6.00
Electric energy for power	4.00
Labor	8.00
Maintenance	3.00
Total	$25.95

Although such an addition to production costs is sizable indeed, the profitability of installing a system must be appraised in view of the additional output and revenue that can be obtained through use of the practice. Assuming that over a number of years corn yields can be boosted an average of 40 bu per acre (using the figure reported in the Iowa test results) and further assuming that the corn can be sold for an average price of $1 per bushel, the additional revenue produced by the innovation would amount to $40 per acre, $24 in excess of the added cost. Under these circumstances, the investment would pay for itself in 4 years. This probably explains why the use of irrigation is expanding rapidly in the Midwest despite the large investment and high annual operating cost associated with it.

Progress in Conversion of Saline Water

Everett D. Howe

A paper presented on Oct. 30, 1957, at the California Section Meeting, San Jose, Calif., by Everett D. Howe, Prof. of Mech. Eng., Univ. of Calif., Berkeley, Calif.

THE production of fresh water from saline water has now been under intensive study in the United States for a little more than 5 years. During this period, investigations of many different phenomena have been undertaken, some abandoned as not sufficiently promising, some carried to the pilot plant stage, and several schemes studied in the laboratory only.

Critical Factors

The two critical factors which limit the applicability of any desalting process are the cost of the water produced and the amount of fuel or power required. The latter is mentioned separately because inroads on fuel resources could become very serious even though fuel costs, as such, might not be too great. The general goal in costs is $100 per acre-ft (or 30 cents per 1,000 gal) which would make demineralized water competitive in price with certain natural water supplies for domestic uses. The goal for fuel used is the heat equivalent of the ideal minimum power reqnirement—approximately 3 kwhr per 1,000 gal of fresh water produced.

The abovementioned ideal minimum power consumption is derived from the laws of thermodynamics and assumes that a separatory device is possible in which all processes are thermodynamically reversible; the temperatures of the product water, waste brine, and incoming sea water are equal, and only a small amount of fresh water is obtained from a very large quantity of sea water. The deviation in the actual devices from these ideal conditions produces thermodynamic losses and results in increased power demands. The degree to which any device can approach the ideal minimum power consumption is uncertain because the most economical mechanically operated device presently developed—the vapor compression distiller—requires a power input greater than 50 hr, or at least seventeen times the ideal minimum. It is generally believed that an approach factor of ten or less should be possible. Further discussion of the thermodynamic reasoning may be found in articles by Murphy (*1*), Gilliland (*2*), and Tribus (*3*).

Research Program

The research program at the University of California (*4*) has been under way since early 1952. Investigations undertaken first at the Berkeley campus were those of the use of waste heat in multieffect distillers, the use of solar heat, and the low temperature difference scheme for combined power generation and distillation. More recently, the use of reversed osmosis and the application of ion exchange to the problem have been under study. At

the Los Angeles campus, there is under study the use of distillation by pressure alone, a study of skimming processes, and a thermodynamic analysis of each of the other investigations.

As may be inferred from the above list, the greatest share of attention at Berkeley has been given to those methods using heat from nonfuel sources—namely, waste heat and solar heat, the latter being available directly or from water warmed in the ocean. The most obvious source of waste heat is that from processing plants and from internal combustion engines and compressors. This heat is available at temperatures ranging from a low of about 90°F for some oil refineries to a high of about 300°F from the exhaust jackets of diesel engines. Heat at the higher temperatures can obviously be used in conventional multieffect distillers. There are, however, considerable quantities of heat available at temperatures too low to be of use in multieffect plants and for this purpose the low temperature difference method seems appropriate.

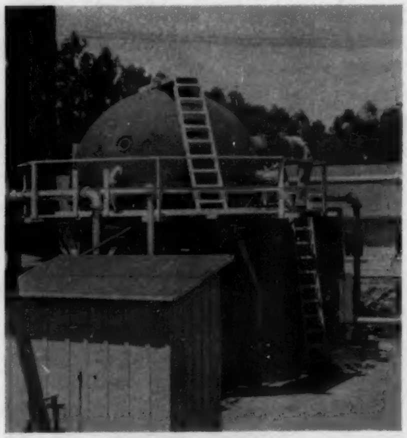

Fig. 1. Low Temperature Difference Plant at Richmond, Calif.

Dome-shaped evaporator has a 12-ft diameter.

Low Temperature Difference

The low temperature difference method consists of vapor production by vacuum flash of warm water followed by condensation in a surface condenser where heat is absorbed by cold water. A turbine wheel can be placed between the evaporator and condenser when the temperature difference between the incoming warm water and cold water supplies is 30°F or more. This turbine wheel can be expected to produce enough power to operate the plant at a large scale.

Two series of tests using this scheme have been run, the first carried out with heat exchangers which were available but not well matched and the second using a plant designed for the purpose. The second plant consists of a dome-shaped evaporator (Fig. 1) with a 12-ft diameter, a turbine wheel located in the downcomer pipe, and a surface condenser located below the evaporator. The plant was designed around the turbine wheel which had been taken from a turbosupercharger obtained from surplus military stocks. Warm water was supplied from a large storage tank to which gas-fired circulating heaters were connected. Cold water was supplied from a cooling tower. The first series of tests yielded information used in the design of the second plant which is being used to explore ways and means of preventing the carryover into the condenser of dissolved air and liquid droplets—the two phenomena which limit the capacity of the plant. So far, the maximum operating rate without appreciable carryover of liquid is about 2,000 gpd (*5*). It is expected that this rating can be increased considerably through

the use of baffles and appropriate nozzles for control of the liquid carryover. Estimated production costs with this equipment are approximately \$100 per acre-ft when temperature differences of 30 F° exist, and about \$170 per acre-ft when temperature differences are only 15°F.

Solar Distillation

Solar distillation seems attractive because solar energy is presumed to be free. Research at Berkeley (*6*) has been concerned with the study of devices using flat-plate collectors without concentration of the sun's rays by lenses or mirrors. The simplest device is the insulated wooden tray covered with sloping panes of glass. The "greenhouse effect" causes the temperature in the interior to rise as high as 150°F during the middle of the day. The glass cover is cooled on the outside by radiation to the sky and by wind motion, and this coolness causes the condensation of the humidity from the air on the inside of the glass. Tests of several units of this type, 4 × 50 ft, have disclosed that 8–10 sq ft of tray surface are required to produce an average of 1 gpd throughout the year. This production rate corresponds to a utilization of about 40 per cent of the incident solar energy. Costs of production with these units are uncertain because of the unknown useful life of the equipment and the variable prices of land. Experience with the plant at the University of California engineering field station at Richmond (Fig. 2) indicates a cost of about \$550 per acre-ft for operation, maintenance, and capital charges on the equipment without the land. Obviously the major component of cost is the capital costs item, so that efforts are currently being directed toward cheaper construction.

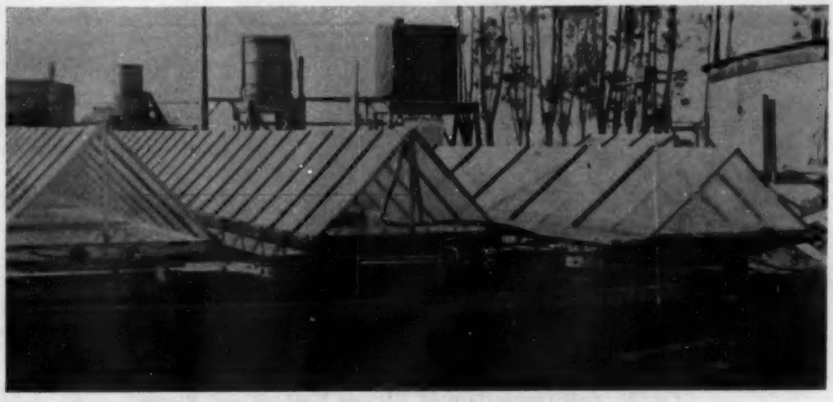

Fig. 2. Experimental Solar Distiller Units

Units, at University of California field station at Richmond, Calif., employ flat-plate collectors.

Osmosis

A third project active at Berkeley is an attempt to use the phenomenon of osmosis. Osmosis is the tendency for pure water to pass through a semipermeable membrane from a more di-

lute solution to a more concentrated one, the membrane forming a barrier between the two solutions. The tendency for flow to occur as indicated is proportional to the difference in concentration between the two solutions and is measurable as a pressure difference. The flow may be stopped by imposing a pressure differential in the reverse direction, and if a sufficiently high differential is imposed the flow can be reversed—that is, the water component of the more concentrated solution will be forced through the membrane into the more dilute solution. This latter process is the one used in demineralizing sea water and is called reversed osmosis. The reversed osmosis scheme may also be thought of as a sieving operation, in which high pressures force the water from a solution through holes which are too small to permit the passage of the dissolved salt ions. Pressures sufficient to cause this type of separation in sea water are in excess of 400 psi. Pores sufficiently small to serve the purpose have been sought in sheet plastics such as cellophane. Experimenters in this area have encountered considerable difficulty in supporting the membranes against the high pressures mentioned.

Capillary Filters

Research work at Berkeley has involved the development of "capillary subultra filters" (*7*). These are bundles of parallel synthetic fibers squeezed tightly together so that the capillary passages along the fibers are smaller than the ions to be screened out. This type of filter unit has also been made of stacks of "washers" made of thin sheet plastic with flow between the washers in a radial direction. Both of these constructions give passages of uniform size. Preliminary results available at the present give promise that these filters will function well as high-strength osmotic membranes. No cost estimates for water produced by

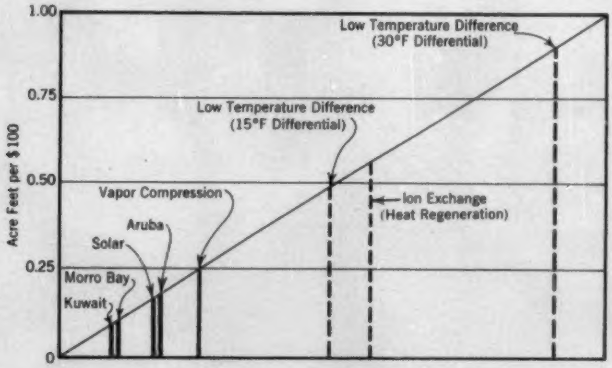

Fig. 3. Cost Comparison of Various Methods and Plants

Production is expressed in fractions of acre-feet obtainable at a cost of $100. Goal is 1 acre-ft per $100. The Kuwait plant is a 4-effect installation; that at Morro Bay, 3 effect; and Aruba, 6 effect.

use of this phenomenon have been made.

Pressure Distillation

Hassler (*8*) has proposed a use of "distillation by pressure alone" which is closely related to osmosis. It is known that the vapor pressure in equilibrium with small drops increases as the drop becomes smaller because of the effect of the surface tension of the liquid on the pressure inside of the drop. By producing half-drops in the very small pores of a plastic membrane by pressure behind the membrane, it is possible to produce water vapor on the "dry" side of the membrane. If then two such membranes with different pressures behind them are placed parallel and a very small distance apart with air between, there will be an evaporation of water from the smaller half-drops and a condensation of the vapor on the larger half-drops. These changes of phase will necessarily be accompanied by the transfer of latent heat across the air gap by conduction through the air. The air gaps contemplated are of the order of 0.001 in. Although the principle has been demonstrated successfully, attempts to construct membrane systems with the required gap have not been successful.

Ion Exchange

Another project currently under way at Berkeley is the attempt to use ion-exchange materials which can be generated with heat rather than chemicals. This method was suggested by Gilliland (*2*). The investigation was undertaken with the idea of using solar heat for the regeneration process. The general scheme is to use a synthetic exchange material which can be regenerated with a solution of ammonium bicarbonate. When sea water passes through a bed of material so regenerated, it exchanges its various dissolved ions for the ammonium bicarbonate. The effluent from the exchange bed is then pure water charged with the latter chemical. This material is then driven out of the water by heat as ammonia gas and carbon dioxide gas. These gases are collected, redissolved, and used to regenerate the exchanger bed.

Work so far on this scheme has consisted of the selection of some of the standard exchange materials which can be used with the ammonia salt and the determination of the number of exchanger beds in series necessary for the complete demineralizing of sea water. Results so far have disclosed at least one exchange material which is suitable and have indicated that five or six stages of ion-exchange beds should be used, with stripping of the gases following each stage. A preliminary design has been made and indicates very low heat requirements, as well as low overall costs at about $180 per acre-ft. The latter figures are uncertain because of a lack of information on the useful life of the exchange materials as well as the very difficult problem of construction materials. It is noted that metals suitable for contact with sea water are not suited for contact with ammonia and vice versa.

Costs

In considering the potentialities of various schemes for demineralizing sea water, it is well to note the several large plants already in use and to compare the costs of these with price and fuel economy goals already mentioned. Cost figures are available for three large plants (*9*) at Morro Bay, Calif., Aruba, in the Netherlands West In-

dies, and Kuwait, on the Arabian Peninsula. These are given in Table 1. Also given in the references cited are figures from which one can deduce the fuel economy in terms of a fuel oil with a heating value of 18,500 Btu/lb. Figure 3 shows the relative costs of water produced in these plants as well as projected costs of some of the undeveloped schemes—all expressed in terms

Figure 4 shows the relative fuel economies in terms of the ratio of fresh water produced to the oil fuel used. An estimate has been made of the fuel required by an eleven-effect distillation plant and these data included for comparison. Solar distillation and the low temperature difference plant are not included on this chart because no fuel is presumed to be involved. The

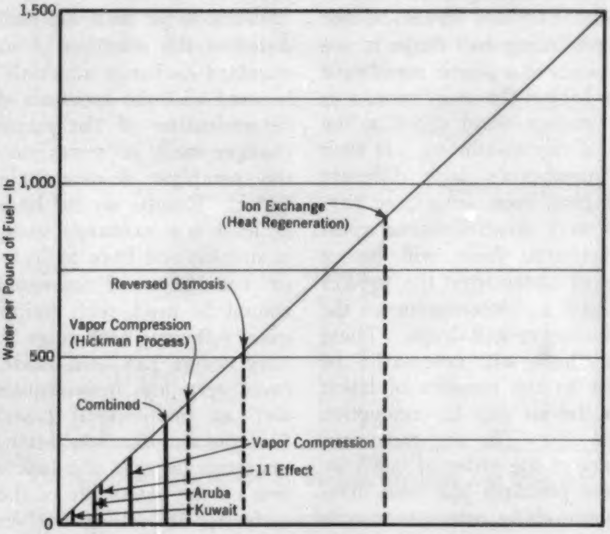

Fig. 4. Fuel Economy Comparison

Water production per pound of fuel is shown for various plants and processes. Goal is 1,500 lb of water per pound of fuel. The Kuwait plant is a 4-effect installation and that at Aruba, 6 effect.

of the fraction of an acre-foot which could be purchased with $100. The apparent high promise of the low temperature difference scheme must be tempered with the reminder that it is of value only where two sources of water are available at different temperatures. Although there are several situations where such differences can be utilized, the method is not universally applicable.

highest economy possible with existing commercial equipment is shown as the "combined plant" and involves a steam power plant which produces power to drive a vapor compression plant. The turbine exhausts into a multieffect distiller plant so that a maximum of water is produced.

Desalting processes may be classified in many ways and will be arranged according to the type of energy

required—namely: heat energy, mechanical work, and electrical energy. If it is presumed that all three types of energy come from fuel, then the amount of fuel required for mechanical work (as from a steam turbine or diesel engine) will be roughly three times the heat value of the work done. Likewise, electrical energy will require fuel which gives about four times the heat equivalent of the electrical energy used. The above ratios are approximate but reflect the values of the efficiency of energy transformation which it is possible to achieve today.

TABLE 1

Costs of Water Produced by Demineralizing Processes

Distillation Process	Location	Cost per 1,000 gal	Water Available at $100 Cost *acre-ft*
Submerged coil 3-effect	Morro Bay	$2.60*	0.118
Submerged coil 6-effect	Aruba	$1.75*	0.175
Four-stage flash	Kuwait	$2.80*	0.110
Vapor compression	—	$1.38†	0.272

* Figures from article "Evaporation Plants Solve Water Shortages (*9*).
† From A. Latham Jr. (*10*).

Heat Energy Processes

Desalting processes which use heat energy as such include multieffect distillation, flash distillation, and supercritical distillation. Multieffect distillation and flash distillation may be considered as one form.

The multieffect principle is that of the reuse of heat—that is, the heat delivered to the first effect is used again and again, as many times as there are effects, and is finally delivered to the condenser.

Multieffect operation is possible because of the variation of the boiling point of sea water with the pressure. Since sea water is a rather complex solution containing a number of dissolved materials, it may be replaced by a sodium chloride solution for purposes of estimation. Figures 5 and 6 show the relation between boiling point and pressure for sodium chloride solutions and the salinity as a parameter. Examination of these figures will give some idea of the temperature and pressure ranges available and the practical limitations of multieffect arrangements.

The amount of water produced by a multieffect plant is somewhat less than the number of effects multiplied by the steam condensed in the first effect, because the latent heat increases as the pressure decreases. For example, a six-effect plant produces about 4.5 lb of water per pound of steam used in the first effect. The goal of 1,500 lb of water per pound of fuel corresponds to 12 Btu/lb of water, whereas the figures given above for the six-effect plant would require about 225 Btu/lb of water. At a boiler efficiency of 80 per cent, the corresponding cost of fuel alone would be $198/acre-ft with heat energy worth 25 cents per 1,000,000 Btu. To approach the fuel goal closely would require the use of over 100 effects—a condition infeasible with shell and tube type heat exchangers.

Another approach to high fuel economy is often sought through the com-

bination of a distiller plant with a power-generating plant. In this arrangement, part of the steam may be charged against power generation, although all of the steam is available in the distiller. In Fig. 3 and 4, the bars labeled "Aruba" pertain to a six-effect plant operated in this manner and give some idea of the gain to be expected.

Supercritical distillation is another approach to low heat demand for high fuel economy. It is called supercritical because the temperatures and pressures used are above the critical values

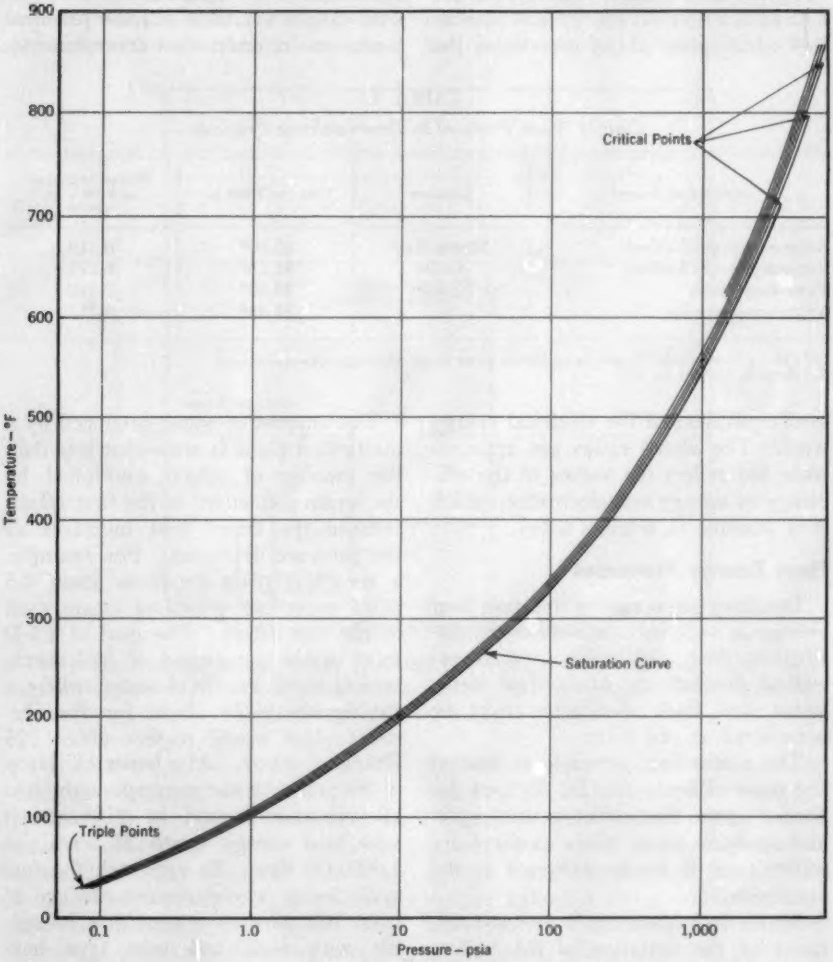

Fig. 5. Relation of Boiling Point and Pressure for Salt Solution

Curves represent, from right to left, salinities of 5, 10, 15, 20, and 25 per cent. Saturation curve is for pure water.

for pure water although they are below the values for sodium chloride solutions as given in Fig. 6. The general idea is to make use of the fact that the latent heat of evaporation of water becomes very small at pressures and temperatures just below the critical values and vanishes at those values. If evaporation and condensation could be carried out very near the critical conditions, the net heat supplied could be very small provided that an effective counterflow heat exchanger is used for heating the incoming sea water and cooling the outgoing brine. It is also necessary to arrange for salvage of the power required to put the sea water under the desired pressure. Experiments on this seemingly attractive scheme were conducted first by von Platen (*11*) and later by Nuclear Development Associates (*12*). Both were discontinued because of the extremely serious problems of corrosion and salt deposition encountered at the high pressures and temperatures in the counterflow heat exchanger. Preliminary tests by von Platen were said to have achieved heat energy requirements as low as 12 Btu/lb.

Mechanical Processes

Under the use of mechanical power for the separation of fresh water from the sea there appear to be three processes in various stages of development —namely, the vapor compression process, the freezing process, and reversed osmosis. The vapor compression process was developed in this country by von Kleinschmidt for use by the US Navy during World War II. In this process (Fig. 7), water vapor is drawn by a compressor from the low-presure (sea water) side of a heat transfer surface, compressed and delivered to the other (high-pressure) side of the heat transfer surface. Because the saturation temperature is higher on the high-pressure side, the latent heat of condensation furnishes the energy to generate more vapor. The most common version of this equipment operates with atmospheric pressure on the low-pressure side and a difference in temperature across the heat transfer surface of about 10°F. Under these conditions, power consumptions of 50–70 kwhr per 1,000 gal have been observed. Cost estimates for water produced by this method quote a price of $1.25 per 1,000 gal ($410 per acre-ft) for installations with a capacity as much as 1 mgd. So far, the largest reported plants are of the order of 200,000 gpd.

Research on the improvement of this method has focused on the possible decrease in the temperature differential across the heat transfer surface. The effect of this would be to decrease the power required by the compressor. It is reported (*13*) that certain experiments on forced-circulation evaporator-condensers have yielded heat transfer coefficients four or five times as great as those possible in the conventional units. Recent developments of a rotating evaporator-condenser by K. C. D. Hickman (*14*) indicate a possible increase in fuel economy which would give a yield ratio of 367 lb of water per pound of fuel as compared to the previous value of 200. Whether or not this fuel saving will be accompanied by decreases in capital and operating costs is not yet clear. Tests on a 25,000-gpd unit of this type are presently being made.

The second method of using mechanical power for desalting sea water is that of freezing by mechanical refrigeration. It is known that the freezing of sea water results in the produc-

tion of pure ice crystals and brine when the temperatures are above the eutectic point. If the brine can be separated from the ice crystals, then the latter can be melted and pure water obtained. Studies of the ice obtained shows that the ice crystals form in clusters and that the spaces within the clusters are filled with the brine. Separation of the brine has been attempted by compression, by washing, and by centrifuging, but no economical scheme has yet been evolved.

From the standpoint of energy, it is noted that the latent heat of fusion is only one-seventh the latent heat of evaporation. The irreversibility due to phase change, therefore, should be less than for distillation. Gilliland (*2*) has assigned one of the greatest fuel economies to this process but there are no reports yet that any experimenter has realized them.

The third method requiring mechanical power is that of reversed osmosis, already discussed. This method is potentially one of high fuel economy but, again, there are no plants to bear out this speculation.

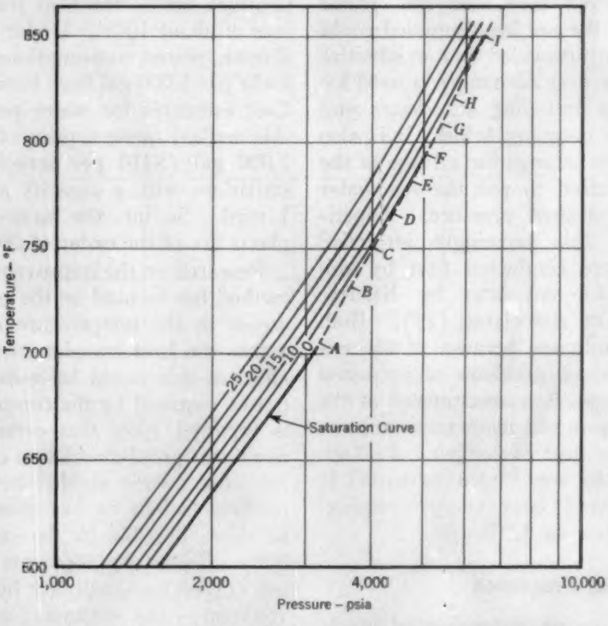

Fig. 6. Boiling Point–Pressure Relationships of Salt Solution

Figures give percentages of salinity. Points A-I *correspond to salinity values of 0–10 per cent.*

Electricity, as such, has been applied in the electrolytic process and in electrodialysis. In both processes, the passage of an electrical current through sea water causes a migration of ions

through the water. In electrolysis the ions may be "plated out" at the electrodes and in electrodialysis the ions pass through electrically charged membranes and are carried away by brines. Electrodialysis equipment is in its early stages of development. Present installations are yielding valuable information (*15*) and should lead to economical plants for saline water with salinity as high as one-seventh that of sea water in the near future.

Conclusion

The use of desalting processes for sea water is presently limited to distillation—either multieffect submerged coil, multistage flash, or vapor compression. Costs of water produced by the multieffect process have been halved during the last 5 years but are still much too high for competition with water from precipitation sources. It is apparent that radical changes in equipment must occur if distillation is to become competitive in cost.

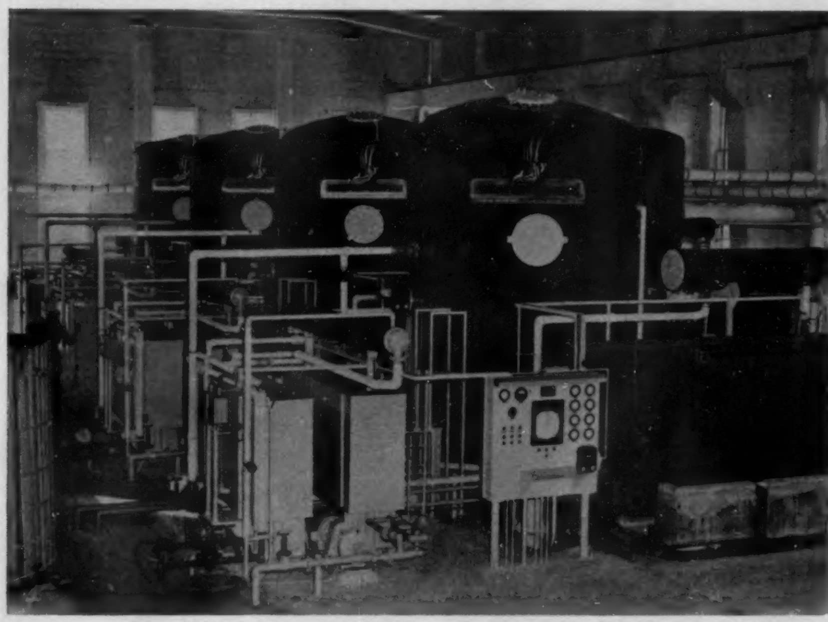

Fig. 7. Vapor Compression Distiller Plant at Bermuda

Plant, which purifies sea water, consists of four diesel-driven units that deliver 50,000 gpd each.

The low temperature difference method for combined power generation and distillation holds considerable promise where large quantities of water at two different temperatures are available.

Solar distillation using simple single-effect glass-covered stills has not been found economical in spite of the "free" solar energy. Further developments

in this area will probably occur in the direction of processes with lower heat demands, such as multieffect devices or ion-exchange equipment using heat regeneration.

The use of mechanical power in reversed osmosis and in mechanical refrigeration seems at this point to merit additional investigation. Energy requirements for these methods are potentially less than for any form of heat-activated distillation.

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Repairing or Replacing Service Laterals

William H. Eppinger

A paper presented on Oct. 31, 1957, at the California Section Meeting, San Jose, Calif., by William H. Eppinger, Gen. Mgr., Orchard Dale County Water Dist., Whittier, Calif.

THE rapid growth in suburban areas which were previously devoted primarily to agriculture, combined with the addition of facilities such as air conditioners, dishwashers, automatic laundries, and two or three baths, and the consumer's ability to pay for greater water use, has placed a heavy demand on the water systems serving such areas. The resulting need for increased pressure to compensate temporarily for undersized mains has placed considerable strain on older water system facilities. Consumers in rural areas, who were previously content with whatever residual pressure was available, were generally satisfied if they had water most of the time, but with the subdivisions of these areas, the new consumers demanded pressures equal to those to which they were accustomed in the urban areas from which they had moved.

These higher pressures on old systems caused numerous leaks in mains and services. The problem of whether to repair or replace a main was relatively simple, because with the exception of large mains, most of them were inadequate to carry the increased flow and a main-replacement program was the best answer. Any repairs made were generally viewed with the idea that they were temporary, and a minimum expenditure was made.

With service laterals, however, the problem of deciding whether to repair or replace was more difficult, because in the majority of situations a small expenditure for a band or clamp was all that was necessary to effect a repair. Generally the service line was at least 30 in. deep and sometimes under pavement, resulting in a sizable labor and equipment charge. Of course, if the pipe was beyond repair with a band, it became necessary to shut off the service line. This could be done by shutting down a section of the main temporarily, which meets with much disfavor from the consumer, by digging out the main connection, or by freezing off the line. Either of the latter two methods is time consuming and more expensive, and inasmuch as the major portion of the excavation for a service line replacement is now already done, it generally does not take much longer to install a new line if it is justified.

Of course, an official should not be too hasty in deciding to replace a line. Often no records are available as to the type of service pipe that was installed. Observation of the pipe at the meter or the leak may not be indicative of the whole line. For example, a short piece of steel pipe or a steel fitting may have been added to a copper service line. In those cities where a good plan for main and service

installations was developed at the inception of the water department, the problem of leaks is generally of no consequence. Usually copper service lines were laid, mostly in the small sizes, and no leakage problems developed except in extreme soil conditions where corrosion attacked the copper or the solder used at the joints. When leaks do develop in properly installed systems, the repairs required are generally minor.

Pressure Requirements

One of the most common complaints received by a water company is that of low pressure at the consumer's premises. Frequently the customer does not register such a complaint until a leak appears and then blames the trouble on the leak. In the California Public Utilities Commission's General Order No. 103 (*1*), certain minimum requirements are established for those private companies operating under their jurisdiction. These standards were adopted after a thorough study by members of the commission and members of the California Section of AWWA. Even though complete agreement was not reached on all points, those portions relating to service pipes and pressures were generally agreed upon. As a result this pamphlet is a good yardstick for all companies whether public or privately owned.

In the order it is stated that the utility shall maintain normal operating pressures of not less than 25 psig at the service connection. A static pressure test may be misleading, in that any obstruction in the line may result in an extreme head loss through friction, causing the resulting pressure to be considerably below the minimum of 25 psig. If a check on the service main reveals that the meter is operating properly and that there is sufficient pressure at the hose bib above the house shutoff, it is apparent that the problem exists in the plumbing of the house itself. It should be explained to the householder that the pressure loss is due to tuberculated piping, too small piping, clogged strainers, a clogged water softener, or a multitude of other reasons which are not the responsibility of the water company to repair. The service man should not be too quick to jump to conclusions, however, because frequently the problem lies between the main and the meter.

Repairing a Lateral

A sure way to prove whether or not an obstruction exists in the service line is to remove the meter, insert a bypass jumper, and check the flow through a test meter discharging directly to atmosphere. The results can be checked by reference to any hydraulics handbook to see if excessive head loss is being experienced (*2*). If it is determined that an excessive pressure loss exists, the service line should be uncovered at the main and inspected at this point. Sometimes it is found that the corporation stop is only partly open. The line can then be disconnected from the main and the flow through the corporation stop checked. In addition to flushing out the corporation stop, it is a good policy to attach a drilling machine and remove any scale, tuberculation, or other obstructions which may be clogging this point. If the corporation stop is clear it can then be assumed that the trouble lies in the lateral.

The lateral can be cleaned to remove any obstructions, but if the pipe is badly tuberculated it is possible that

leaks can be caused by such cleaning, especially if the pipe is very old (*3*). If such is true and the pipe is 1-in. standard or larger, the possibility of lining with either copper or plastic tubing of the next smaller size can be considered. Because either copper or plastic tubing has a Hazen-Williams *C* factor of 150, such relining can be accomplished without an appreciable loss in head over the original piping. Some method of mechanical cleaning should be used, in order to remove the hard obstructions that may injure the copper or plastic tubing. A prover should be finally used to assure a large enough opening to pass the new liner. The new liner can then be attached to the existing corporation and curb stops by iron pipe and tubing adapters. Generally, the use of flared connections is the most expedient method of making the attachment. If the line cannot be opened sufficiently to pass the liner or if computation shows that the liner will develop too much head loss, the only alternative left is to push a new service through alongside the old one.

Electrolysis

If it is determined upon exposure of the line that electrolysis is the cause of the leak and the service line is of steel pipe, it can generally be assumed that a repair will be of a temporary nature only. In those areas where steel pipe was used for service lines and no trouble was encountered, the installation of steel pipe was continued because of the savings involved. With the advent of houses with concrete floors, however, it was the general practice to install copper piping under the concrete slab. Such practices created a corrosion problem if the copper was attached directly to a steel line, or if any steel was placed in the system between the house and the main. Steel fittings should not be used in connection with copper service lines without some type of insulation. The author has observed complete disintegration within 2 years time of galvanized tees used in conjunction with copper lines. Once a steel service clamp on asbestos-cement pipe connected to a copper service line was completely eaten through in 2 years time. A piece of new steel pipe installed in an old line will corrode faster, due to dissimilarity of metals (*4*). If it is necessary to connect dissimilar metals to each other, it would be wise to use some type of insulating bushing or connector, although properly wrapped steel pipe is resistant to external corrosion. Cathodic protection can also be employed to prevent corrosion, but if protection methods are to be employed, an economic study should be made to determine whether it is more feasible than using copper or plastic pipe. Subsequent installation of insulating bushings might help, but by the time leaks start to develop, it is generally too late to arrest the action.

Replacement of Laterals

Where pipeline failures are occurring frequently, public relations in the area suffer considerably. The consumers discuss the problem and decide that the system is failing, and considerable time is spent explaining the reason for the failures. If it has been decided to replace such lines rather than repair any ensuing breaks, a considerable saving may be effected by doing such replacements on a quantity basis, allowing shutdown of the entire block with a resultant saving in labor. The plan can be explained in a short letter to the consumers in the area affected, resulting in a gratifying boost

in public relations and real cooperation from the consumers. The work can be done during a slack period when the service crew is not otherwise occupied.

If the original services were installed before street improvements were made, it is likely that the meters were placed in poor locations for reading and servicing. In those areas where the climate allows the meter to be set at the curb, it would seem most desirable from the water company standpoint to move the meter. If a service lateral replacement program is to be put into effect, relocation of the meters can be correlated with such a program.

A check of various water companies shows that the cost of service installations varies considerably. A completely new service including meter, costs from $75 each for ¾-in. services to $300 each for 2-in. services. The labor and equipment costs on the smaller services are approximately 50 per cent of the total cost and proportionally less on the larger sizes. The time required either to install a new service or to repair an old one can vary considerably with the type of soil encountered and the depth required. The author's experience with water services has been in the Southern California area, where freezing of service lines is not a problem; hence, the cover required is only that sufficient to meet local road department requirements. In those areas where the lines are much deeper, it would appear that the total cost would be proportionally higher. As the ratio of labor and equipment cost versus material cost increases, it would appear more feasible to replace than repair. It is evident that because of the high relative cost of labor and equipment, if a replacement is made, material with the longest life be used to avoid high repair costs. The labor involved in repair or replacement can often be many times the price of bargain materials.

Conclusion

This article has endeavored to point out the various hidden factors that are encountered in failing services and to give some items to consider before undertaking a repair job. Generally, the water supervisor has to live with his mistakes, and if he can install new services and either repair or replace the old ones so they give maximum service, he has developed a system of which he can be proud.

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Proper Methods for Breaking and Repairing Pavement

Joseph M. Rogeven

A paper presented on Sep. 26, 1957, at the Michigan Section Meeting, Detroit, Mich., by Joseph M. Rogeven, Supt., Water Dept., Jackson, Mich.

PERHAPS one of the greatest causes of contention between utilities and the street departments of the public works group is the question of pavement breaks and repairs, and it appears that it will continue to be until the utilities can get their workmen on the job to make proper openings, backfill with proper materials, and place the backfill in the excavation properly. All of the fault does not lie with the utilities, however, because once a permanent repair is made to the pavement cut, it must receive proper maintenance whenever necessary. If the repair is left without crackfilling at least, trouble is likely to occur.

This article discusses the subject in five steps: [1] opening of the pavement, [2] excavation, [3] backfilling, [4] repair, and [5] maintenance. If any one of these steps is done improperly or haphazardly, the rest of the job will not hold up, and the whole job will become as bad or even worse than the step that is done improperly. Not one step can be omitted if a good job is to result.

Opening and Excavation

The first operation—opening of the pavement—must be done with the use of proper tools, in order to provide a neat and smooth edge to finish. Improper tools make irregular edges which are difficult to maintain (Fig. 1). Asphalt surfaces should be cut through with a regular asphalt cutter and not a wedge point or pavement-breaking point. The straighter and more vertical this first cutting, the better will be the repair job and the longer it will last (Fig. 2).

If there is a concrete base under the asphalt, this must be taken out with a hand-operated pavement breaker. Heavier equipment does a fine job if the entire pavement is to be removed or if an edge is cut along the desired ditch opening of sufficient depth to prevent the fracture from going beyond the ditch line. If there is reinforcing rod in the concrete, naturally that must be burned off, or if wire mesh it can be cut with bolt cutters.

Care must be taken when breaking the concrete to avoid long fractures or breaking too much out at a time. Such a fracture in the concrete will become a source of trouble as traffic rolls over it and as extreme temperatures at various seasons have their effects on it, the joint opens up, water gets in, and it is just a matter of time before the repair settles down and needs to be removed again.

It is recommended by many engineers and street maintenance operators that all pavement openings be 1 ft wider on all edges than the bank of the

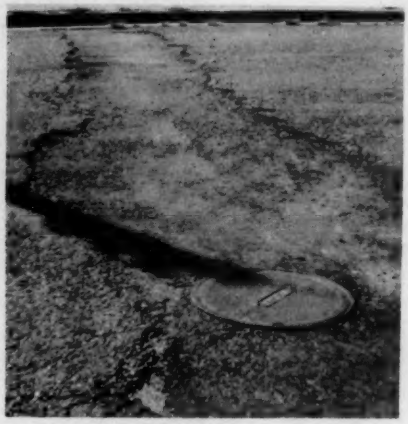


Fig. 1. Results of Improper Pavement Opening

This picture demonstrates what can result when cuts are made with improper tools, making irregular edges in the pavement.

ditch. This is an excellent thought and works out very well on paper and in soils that contain a lot of clay. But in many localities this is next to impossible, because ground and soil conditions will not support such a bank. In fact, sometimes there are ditches that won't even stand up under a 1:1 slope.

After the opening is made, the next step is excavation. After a few years of experience in a particular locality, a workman will know what to expect in the way of ground conditions. If he finds that the excavated material is of suitable quality for backfilling, then he is in luck; he will not need to haul the material away. If he finds that it is of inferior quality for backfilling, however, then he might just as well get rid of it at this stage, to eliminate

Fig. 2. Proper Method of Pavement Cutting

The straight edges along this cut make it more likely that the repair job will last.

Fig. 3. Tamping the Backfill

This gang tamper provides a good method of compacting the backfill material properly.

the problem of maintaining the cut at a later date.

If the excavation is deeper than 6 ft and the banks are loose, much time can be saved at a later stage, as well as better protection provided for the workmen, if the trench is sheeted. The amount of sheeting necessary depends on the stability of the soil. If the banks are permitted to cave in and out from under the pavement, the entire area of concrete or blacktop over the settled area will eventually settle and will have to be replaced. It may take a month, a year, or perhaps even 20 years, depending on the workmen and the ground conditions. The more uniform and regular an excavation, the better the backfill will stay in place, and less settling of the cut will be experienced.

Backfilling and Repair

With the hole completed and the connection completed, the most important phase of the project begins, that of backfilling. This can be accomplished in many ways, and sometimes it is seemingly impossible to eliminate settling, in spite of what is done.

First, the materials must be compactable, such as damp sand or gravel, sand, and clay. Any aggregate that would make good concrete or road gravel will make good backfill material. This material can be placed in 6-in. layers and hand tamped or power tamped, or in thicker layers up to 1 ft, depending on the material and the kind of tamping equipment used (Fig. 3). Too little emphasis has been placed on the importance of placing the aggregate and the tamping process, but it does

Fig. 4. Pavement Repair

This section of pavement has a concrete base (light area), which has been cleaned and prepared for application of the asphalt surface (dark area), which must be made smooth.

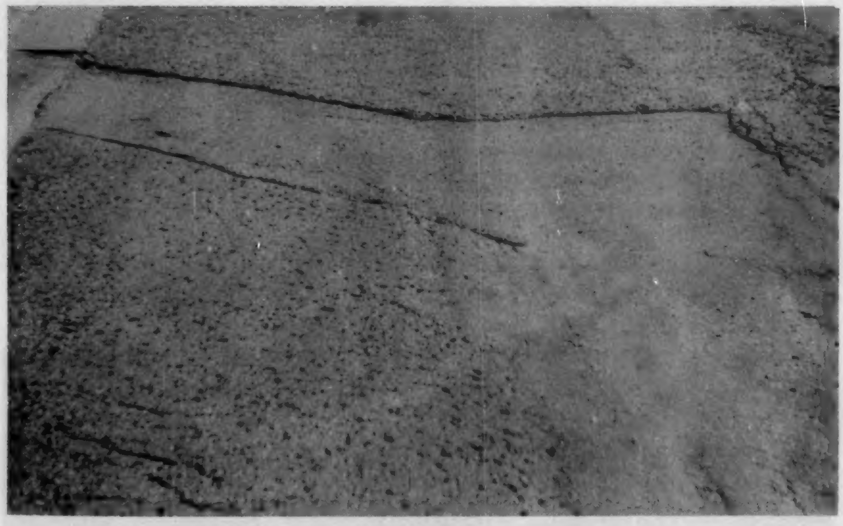


Fig. 5. Example of Poor Maintenance

Lack of proper crackfilling has caused this condition.

not take long before the effects of a poor tamping job show up. In writing specifications on backfilling, it is often the policy to tie down the contractor to exact and proper backfill methods and then go out on the job and let the men simply push the backfill material in the trench and run the tracks of the loader or bulldozer along the top of the fill.

Another method of backfill compaction which is very successful is that of flushing in the material. This is not filling the ditch with water first and then pushing in the fill, but rather a method of placing the fill in the ditch up to the halfway point and then running water on to the fill directly from the top, moving back and forth along the ditch until the water just barely begins to stand; the ditch is then filled to the three-quarter point, the flushing process repeated, and then the last portion is tamped. This will enable the men to make use of the area immediately, whereas if they flush to the top of the ditch, they may use too much water and find that facilities will have to be barricaded for some time until it dries out.

There are many kinds of power tampers on the market today, but they do not eliminate the need of carefully placing the backfill materials and the use of proper aggregate.

After the trench has been backfilled to the proper height, it is necessary to make the repair. This alone will cost from \$3.60 to \$12 per square yard, depending on whether the surface is blacktop, concrete, or brick. In any event, whatever type of surface repair it is, care must be taken to be sure that all loose edges or unsupported edges of the cut are removed. The finished surface must be smooth to

avoid any bumping from the repair to the original pavement, and a liquid crack filler must be poured along the joint between the old and new surfaces. Figure 4 shows two stages of repair.

From all appearances it would seem that the job is completed, but the next spring after the frost comes out of the ground, pavements often begin to heave and repair crews are sent out to make necessary repairs to the previous year's pavement cuts. When particular notice is taken of the ones that have settled, it is seen that of the properly backfilled ditches only a very few of them will need any attention, whereas those ditches which were poorly backfilled will require several treatments before they stop settling.

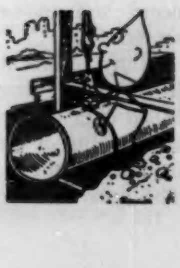
Maintenance

Maintenance of the cut cannot be neglected without bad results (Fig. 5). Maintenance refers primarily to stopping the flow of water into the joint between the repair and the original pavement. This is accomplished either by crackfilling or resurfacing. Crackfilling is merely the pouring of the asphalt from a small can into or along the joint. On a large scale, special nonhardening crackfilling asphalts are available and special crackfilling ladles. Small 1-pint cans, available at builder supply stores, can be kept on hand for small cuts and on-the-spot maintenance.

If it is necessary to resurface the cut, it is better when possible to cut out the repair to make the cut from 6 in. to 1 ft larger all around than the original opening was. This will stagger any joints and provide greater support to the new repair.

Summary

In summary, it should be pointed out that the five steps of making proper pavement breaks and repairs are all equally important. Not one of these can be omitted or slighted if the whole job is to be accomplished satisfactorily.



Water Hammer Allowances in Pipe Design

Committee Report

A report of Committee 8110 D—Allowances for Water Hammer, presented on May 13, 1957, at the Annual Conference, Atlantic City, N.J., by S. Logan Kerr (Chairman), Cons. Engr., Flourtown, Pa. Other members of the committee were: Arthur J. Maahs, Arthur E. Schuh, Hugh F. Kennison, Walter H. Cates, Samuel M. Clarke, Vance C. Lischer, Fred G. Gordon, and Richard E. Hemborg.

THE starting point for all investigations of water hammer allowances in water works piping seems to be the schedules prepared in the 1890's by Dexter Brackett. These found their way into the New England Water Works Assn. standards for cast-iron pipe (*1*), were continued in the 1908 standards of the AWWA (*2*), and in committee standards of the American Standards Association (*3*).

A report in 1933 by A. V. Ruggles of the Cast-Iron Pipe Research Committee listed a number of cases of pipe breaks due to water hammer. The conditions varied widely from one installation to another. The general conclusion of the research committee was to retain the standard allowances for water hammer in cast-iron pipe, since no acceptable substitute was offered.

A broad review of the problem was attempted in a panel discussion "Standard Allowances for Water Hammer" held at the 1952 AWWA Annual Conference in Kansas City, at which viewpoints of consultants, operating engineers and manufacturers were presented (*4*).

Many technical articles on the subject of water hammer have been published over the last 25 years under the sponsorship of the committee on water hammer of the ASME, among others. The basic theory has been well established, but the application in detail to water works distribution systems has proved to be a most complex undertaking, and very few such investigations have been carried out.

Plan of Operation

The first meeting of this committee was held Nov. 16, 1955, in Chicago, Ill., and a plan of operation set up. One basic objective was to formulate some policy relating to the so-called standard allowances for water hammer that could assist committees of AWWA charged with preparing standards for pipe. The special task committee was charged only with formulating a policy and was not to prepare exhaustive analyses of pipe design standards.

The functions of the existing standards committees were not to be superseded, nor was it the function of this committee to prepare a detailed design manual on distribution systems or to set up a permanent committee on the subject.

It was at once apparent that large-diameter transmission mains, pump discharge lines, and trunk mains usually had special design problems. Investments in such conduits were sufficiently large to justify detailed engineering and economic studies of these problems.

Operating requirements varied as did many other factors. It was felt that no single rule or sets of rules could be put forward that would cover all or even a majority of situations.

It was considered feasible and most desirable to concentrate on distribution system piping, thus covering the bulk of specification purchasing. Here again the problem had many aspects, such as size, material, pressure ratings, factors of safety, and operating conditions. A size limit of 18 in. diameter and under was fixed—maybe in a somewhat arbitrary manner—but the other items were left wide open.

Each member of the committee agreed to undertake a specific assignment. Fred G. Gordon was asked as a practical water works man to collect any statistical data available on breaks in distribution systems and to learn, if possible, how many failures were due to water hammer. Vance C. Lischer undertook a survey of the effect of quick-acting valves on the consumers property. Automatic laundries, automatic bottling plants, and other types of installations were found to be sources of trouble due to water hammer.

As to the basic design of pipe, each of the four industry representatives was asked to review his standard designs and take particular notice of factors of safety above the pressure ratings of the pipe. They were asked to review statements made at the Kansas City convention concerning the effect of water hammer on pipe strength (*4*) and to indicate whether or not a standard allowance for water hammer was either necessary or desirable.

Other factors were investigated, rumors run down, and a host of items studied with reference to distribution line failures that might be influenced by water hammer. Information was obtained from a wide variety of sources, and excellent cooperation was given to the special task committee by all who were called upon to assist in the work.

It was agreed that all minutes, reports, and other data would be held confidential within the committee pending a final report. Formal approval for the release of all material to be used herein has now been secured from the municipal water departments, from private water companies, and the respective trade or commercial organizations representing the pipe manufacturing industries.

Results of Investigations

Appendix 1 gives the condensation of information received on breaks in distribution systems from a number of sources. With cast-iron pipe being so widely used over such a long period of time, it is natural that statistical data should include more references to it. The length-of-service records and the reports received gave only a limited coverage of other types of pipe, but the conclusions drawn were the same.

Appendix 2 gives the results of a survey of consumers' operations, insofar as they cause water hammer in distribution systems or in neighboring service connections.

Appendix 3 contains the statements of the four pipe manufacturing industries with respect to the design of pipe and its ability to withstand water hammer pressures.

Interesting information was gathered by all members of the committee and discussed at the meetings in St. Louis, May 1956 (*5*); Chicago, December 1956; and Atlantic City, May 1957.

A brief summary of a few of these items and the conclusions reached are:

1. The subject of raising pressures on systems after operating at a "below rating" level for some time is a fascinating one for speculation. Some actual experiences were investigated, but no general rule could be found, except that maintenance work almost always increased after pressure levels were raised. Failures seemed to happen rather soon after the pressures were increased—usually within 1 day to 1 week. Some failures occurred later. Many leaks occurred in service connections, rather than in the distribution lines. Within a 6-month period, the weak spots had usually been corrected, and the higher pressures were maintained as a permanent condition.

2. The pressure classification of pipe, particularly cast-iron pipe installed many years ago, was found to be a potential hazard. In the period from 1900 to 1910, letter designations varied. Under one standard, the lightest weight of pipe, Class A, meant 50 ft of head, and under other standards it meant 100 ft. Class B called for 100 ft and 200 ft respectively, and other classifications varied accordingly in increments of 50 or 100 ft. In a different system, the heaviest pipe was denoted as Class A, roughly equivalent to the AWWA 1908 designation of Class F. Class F in this reversed terminology is about the same as AWWA Class C.

Further complications come from so-called "city standards" or "manufacturers standards," quite commonly used about 50 years ago. The casting position also had an effect, with floating cores being troublesome when horizontal molds were used. In the period from 1900 to 1910, so many different standards were in effect that the user of such pipe would do well to check his records as to what was actually supplied. Steel pipe with allowances for corrosion and other factors introduced problems in evaluating the need for water hammer allowances.

3. The strength of joints, the jointing materials, and the design of joints with or without mechanical locking or tie rods also were reviewed.

4. Lack of proper anchorage against thrust at dead ends, elbows, bends, and branches reduced the overall system safety factors, and failures occurred with relatively small increases in pressures.

5. The effect of fire service was also studied. One favorite question was "What happens when a fire hydrant slams shut?" The answers were diverse, but deliberate slamming seemed to be a problem of education, whereas accidental slamming, due to broken mechanisms, was a matter of maintenance. The effect in either case was undesirable, as line velocities were high and heavy potential pressure surges were possible.

Nature of Water Hammer

Exploring the nature of water hammer leads to certain popular conclusions: [1] it is usually associated with slam, hammer, or noise; [2] it is a sudden blow; [3] it causes a rupture of the pipe; and [4] it is a great mystery.

While one or more of these four factors may be present, water hammer does not require them all in every case. Many times the water hammer surge is noiseless, but still large increases in pressure are present. Water hammer is not always a sudden blow, but dangerous surge pressures can rise at a rate easily recorded on a gage. It may not rupture the pipe at all, but may cause progressive failures of some types of joints, shifting of the pipe due to poor anchorage, or other manifes-

tations that would accompany any increase in pressure. In some cases, water hammer of smaller magnitude at repeated intervals can stir up the sediment in the lines and cause consumer complaints.

Water hammer is a pressure change caused by a change in flow. The more rapid the change, the greater the pressure variation. The corrective measures are simple: to control the permissible rate of change of flow to safe limits, or to provide relief means that will be fully effective when needed. Only if neither of these solutions is possible should added strength be used.

Conclusions

It must be admitted that distribution systems have survived service conditions amazingly well. Perhaps this is due to the presence of multiple factors of safety.

It must also be admitted that the basic causes of pipe failures may include water hammer. A break in a pipeline usually destroys the evidence, or at least a part of it. The absence of detailed data on the pressures, operating procedures, and other conditions immediately preceding the failure makes it difficult to fix the definite blame in all cases. Laying and handling conditions often impose greater strains on pipe than do water hammer occurrences. The same is true of traffic, maintenance, and soil factors.

The more data that can be obtained on the conditions attending a pipe failure, the better understanding can be reached as to its cause. The understandable haste in making repairs and in restoring service after a break, however, often omits a check upon conditions that led up to the failure. There is a great scarcity of test data on surges in distribution systems. More information on this subject should help to correct or avoid water hammer problems.

The greater use of sensitive recording pressure gages at various points in distribution systems could aid in tracing the causes of many failures and prove whether or not they are due to water hammer. The fundamental accuracy of water hammer theory has been established. The problem of application to distribution systems, however, is complex. Electronic computing devices have proved a valuable tool in solving flow problems. They may in the future be a means of predicting surges in distribution lines with a reasonable degree of accuracy.

All too frequently, a factor of safety is added to a design allowance on top of a margin to cover other factors of safety. These all tend toward a recommendation for an overall factor of safety to cover the entire range of adverse conditions, although the individual pipe manufacturing industries have individual opinions on such a proposal (Appendix 3).

One thing was agreed upon—the need for a valid definition of distribution system design and operating pressures. With such a basis fixed, a minimum weight or strength of pipe could be established. The allowance added to allowances (or factor of safety added to factor of safety) could thus be eliminated to a great extent.

Recommendations

The special task committee unanimously recommends:

1. That a clearer basis of setting distribution pressures for design and operation be established.

2. That extraneous, consumer-caused disturbances be remedied by the consumer.

3. That consumer-caused water hammer should not be made the reason for heavier distribution systems.

4. That the factors of safety in the pipe itself be considered adequate to cover water hammer as well as other factors, such as handling, laying, traffic, and trench loads *provided the conditions of service are properly established.*

5. That some agreement be reached on a minimum weight or pressure rating to be used in selecting pipe for distribution systems.

6. That, with pressure ratings to be established as recommended in Items 4 and 5, no separate allowance for water hammer be made for distribution piping 18 in. and under.

Since Items 2 and 3 require protective clauses in service agreements or contracts, the detailed wording should be subject to legal opinion. Several suggested clauses were prepared, but the committee felt such matters were beyond the scope of its assignment.

References

1. Standard Specifications for Cast-Iron Pipe and Other Castings. *J. NEWWA*, **16**:330 (1902) & **17**:84 (1903).
2. AWWA Standard for Cast-Iron Pressure Fittings—AWWA C100-55 (formerly 7C.1-1908). Am. Wtr. Wks. Assn., New York (1955).
3. American Recommended Practice Manual for the Computation of Strength and Thickness of Cast-Iron Pipe—ASA A21.1. Am. Wtr. Wks. Assn., New York (1939). *Jour. AWWA*, **31**: *following* 2194 (Dec. 1939).
4. KERR, S. L., ET AL. Standard Allowances for Water Hammer. *Jour. AWWA*, **44**:977 (Nov. 1952).
5. DOLSON, F. E. Water Hammer Problems at St. Louis. *Jour. AWWA*, **48**:1085 (Sep. 1956).

APPENDIX 1

Survey of Failures in Distribution Systems

A report presented by Fred G. Gordon, Asst. Chief Engr., Dept. of Public Works, Chicago, Ill.

At an organization meeting of the committee headed by S. Logan Kerr it was suggested that a survey be conducted to discover, if possible, the causes of and the details of failure reputedly caused by water hammer. It was also agreed to review statistics in Chicago having to do with main breaks and to secure similar information from certain other representative cities.

The Chicago data (from the reports of the Water Distribution Division) covered a more or less detailed description of breaks in mains 24 in. and larger and a summary by size and number of breaks in mains smaller than 24 in. A 20-year period was reviewed. Of the 62 breaks in the larger mains during the 20 years, one was ascribed to water hammer. This break appeared definitely due to the improper operation of a vertical triple-expansion pumping engine in a station located near the broken main. Other breaks in large mains consisted of circumferential cracks, longitudinal cracks, and combinations of the two. Seventy-four per cent of all cracks occurred in the fall and winter, while all of the circumferential cracks occurred during this same period. Only one circumferential crack was reported to extend entirely around the pipe; most of them were confined to a quadrant.

The relationship between main diameters and annual breaks per 100 miles of pipe was studied. For 16–48-in. sizes inclusive, the average number of annual breaks per 100 miles of

main was approximately one (0.86 to 1.30) for each size of main. Below 16-in. diameter, however, the number of annual breaks per 100 miles of main increased rather uniformly: 1.5 for 12-in.; 2.5 for 8-in.; 6.0 for 6-in.; and 14.0 for 4-in. pipe. The conclusion drawn was that there was no evidence of failures due to water hammer, but that the increasing number of main breaks with smaller sizes supported an assumption of failures due to beam action.

Other Reports

A. C. Michael, assistant chief engineer, Department of Water Supply, Detroit, furnished the committee with data similar to those secured at Chicago. In that city for a 10-year period the number of breaks annually per 100 miles of pipe averaged 1.6 for mains 24–48 in. Below 24 in. the number of breaks per 100 miles of pipe averaged 3.8 for 16-in.; 7.2 for 12-in.; 28.2 for 8-in.; 23.6 for 6-in.; and 18.0 for 4-in. pipe. Here again the pattern of an increasing number of breaks for decreasing diameters of mains is evident. He called attention to a discussion by Lawrence G. Lenhardt of Detroit in the November 1952 JOURNAL on water hammer in the distribution system (*1*). The author had referred to breaks in 6-in., 8-in., and 12-in. mains as being caused by flushometers and other types of quick-closing valves.

Use of fixed and portable recording pressure gages over the system permits detection of unusual pressure variations which are traced to the source and eliminated.

H. W. Niemeyer, superintendent of distribution at Indianapolis, furnished details of main breaks from 1926 to 1955 in the company's distribution system. The annual number of pipe failures for all sizes of pipe 2–42 in. averaged 2.58 per 100 miles of pipe. In a list of causes, water hammer was not included. Mr. Niemeyer said that "while minor surges may have been a factor in some failures, we have not had a single experience in which a break could be directly attributed to such a cause."

Gerald E. Arnold, general superintendent of the Philadelphia water department, stated that breaks caused by water hammer were confined to discharge lines from pumping stations and that conditions causing them had been corrected. He said that "in general, the pressures in our distribution system are low enough that minor shocks do not give us serious trouble."

R. E. Hemborg, engineer of water distribution, Los Angeles, reported 534 leaks in 4,674 miles of cast-iron mains for the fiscal year 1955–56, or an average of 11.4 per 100 miles. There were 3,849 leaks in 873 miles of steel pipe, an average of 440 per 100 miles. Forty-five leaks occurred in 236 miles of asbestos-cement pipe, an average of 19.1 leaks per 100 miles. He stated that most of the leaks in the cast-iron and steel pipe were due to aggressive soils in parts of the city and that measures had been taken since the mid-1930's to combat this condition by encasing new cast-iron pipe with a minimum of 4 in. of sand and using coal-tar enamel or cement-mortar coating or both for steel pipe. "Most of the breaks in asbestos-cement pipe have been occasioned by the activities of other utilities . . . although there have been some breaks occasioned by heavy construction equipment rolling over freshly flooded ditches. " Recent practice is to install "asbestos-cement pipe with about a foot greater cover than other types of pipe."

In Los Angeles, Class 250 cast-iron pipe is used up to and including 12-in. diameter; above this size either Class 200 or Class 250 is used, depending on water pressure. No cast-iron pipe is installed above 24-in. diameter. All of the asbestos-cement pipe installed is Class 200. All of the steel pipe installed from 4 in. to 36 in. varies from $\frac{1}{4}$-in. to $\frac{3}{8}$-in. plate thickness. Any pipe above 36 in. diameter has a plate thickness designed for a unit stress of 13,500 psi, with a minimum of $\frac{3}{8}$-in. plate.

There are more than 100 different pressure zones in the Los Angeles system, the majority served by pressure regulator stations equipped with relief valves. The relief valves guard against surges or regulator failures.

Water hammer problems due to quick-closing automatic valves have been experienced. These may not damage the distribution system but can cause damage to facilities of adjacent consumers.

Hemborg summed up his report by stating that it was his belief "that the minor amount of damage to the water distribution system of the city of Los Angeles, which . . . is also true of a good majority of the water systems in the United States, is occasioned by the fact that most pipelines are not operated at the full designed water pressure, coupled with the fact that there is usually an ample factor of safety in the material used. This is particularly true of both cast iron and asbestos-cement, gate valves and fittings . . . there is considerably more damage to house plumbing, particularly water heaters, occasioned by quick-closing valves."

Meyer Serkes, distribution division engineer, St. Louis, compiled a record of main breaks in that system for the years 1934–55. In the cast-iron pipe up to 36-in. size, breaks per 100 miles per year averaged 31 for mains smaller than 6-in., four to five for sizes 6–10-in., and approximately two for 12-in., 20-in. and 30-in. diameters.

Breaks in steel mains from 36-in. to 62-in. diameter ranged from as low as three to as high as 59. Steel line breaks are from pitting. Mr. Serkes did not furnish any specific instances of breaks from water hammer.

In summary, it may be said that the information obtained from Detroit, Indianapolis, Philadelphia, Los Angeles, St. Louis, and Chicago does not indicate that water hammer is responsible for main breaks in the distribution system, except in infrequent and readily recognizable instances.

Reference

1. Lenhardt, L. G. Discussion—Standard Allowances for Water Hammer. *Jour. AWWA*, **44**:993 (Nov. 1952).

APPENDIX 2

Effects of Quick-Acting Valves

A report presented by Vance C. Lischer, Partner, Horner & Shifrin, Cons. Engrs., St. Louis, Mo.

Although it is conceivable that an occasional severe water hammer disturbance might result from a mishap in a large water-using industry, it is more reasonable that conditions requiring attention and remedial measures will occur in those industries using equipment which by its nature requires quick-opening and closing valves. Such equipment is used in laundries,

process industries, and in bottling plants and breweries. In these industries certain hydraulic functions will be repetitive and, if detrimental effects are produced, such as pipe or equipment failures or noise, the industries or manufacturers will have become aware of the problem. There should be established some pattern for correction.

In order to appraise the potential for surge or water hammer on consumer premises which might be propagated to the mains in the water system, and to determine what remedial measures, if any, were being made, questionnaires were sent to four large water-using industries in the St. Louis area, to thirteen manufacturers of laundry equipment throughout the country, and to a laundry consultant. Helpful replies were received from all four of the industries, six of the laundry machine manufacturers, and from the consultant.

Large Industry

Two large breweries reported that barrel and bottle washers use large quantities of water and are equipped with quick-opening and closing valves. These machines have valves operating simultaneously, and several such machines are installed in parallel. One brewery has eighteen machines with a total water rate of 2,400 gpm. Simultaneous operation of valves is possible but unlikely. The cycle of operation for individual washing valves or banks of parallel valves is 5 sec open, 4 sec closed. The other brewery reports 2-in. rapid-operating valves delivering 100 gpm each with 13–20 operations per minute.

Water hammer and noise have been experienced. One brewery using mostly welded pipe reports no failures. The other brewery reports failures. Corrective devices used consist of air chambers and snubbers. Of two large chemical companies questioned, one reported that it has some quick-operating valves, uses air chambers, and has had no failures. It gave no data on rate of flow or pressure rise. The other company reported that quick-operating valves are not commonly used, and that it has not experienced serious problems.

Laundry Equipment Manufacturers

The questionnaire to the laundry equipment manufacturers requested data on the following items: [1] type of valve operation—manual, solenoid, air, hydraulic, electric motor; [2] size of valve; [3] speed of operation; [4] frequency of operation; [5] rate of flow interrupted; [6] type of surge suppressors or controls used, if any; [7] possibility of simultaneous operation of several valves; [8] pressure required; and [9] surge test data, if available.

TABLE 1

*Data on Rate of Flow Interrupted**

Valve Sizes *in.*	Rate of Flow *gpm*
$\frac{3}{4}$, 1	35– 50
$1\frac{1}{4}$, $1\frac{1}{2}$	60–120
2, $2\frac{1}{2}$	65–300
3	300–600

* Based on replies from three manufacturers of laundry equipment.

The questionnaire was answered by five firms. A sixth firm gave some useful data, but did not answer the questionnaire. Results show that valve operation is manual, solenoid, hydraulic, or air, and valve sizes vary from $\frac{3}{4}$ in. to 3 in. Speed of operation is reported from "instantaneous," 1-sec opening and closing, to "adjustable," up to 4 sec. Frequency is given as 80–100 operations per 8-hr day.

The rate of flow interrupted for various valve sizes is shown in Table 1. Surge control facilities were not included as standard equipment. The companies recommended air chambers, snubbers, or adjustment of the rate of operation where possible. One manufacturer recommended rubber hose for a connection to supply. Simultaneous operation of parallel machines was reported possible.

There was little response to the question regarding pressure required. One firm indicated that pressure as low as 15 psi was satisfactory. One stated that pressures of 75–100 psi were undesirable as "water hammer is a serious problem," but that 40 psi was adequate. Several recommended the use of pressure-reducing valves.

Surge test data were not given, although three firms stated they had run such tests. There apparently was little understanding among the manufacturers of the principles of water hammer, judging from the character of answers given. One manufacturer had recognized the problem by furnishing maintenance and service instructions to its dealers, suggesting several snubber devices for water hammer corrections.

The laundry consultant reported that, in his experience with the problems of laundries, hotels, and institutions, water hammer has been a problem. It has evidenced itself in damaged valves, pumps, and other appurtenances. He mentioned the use of air chambers but not any of the proprietary snubber or suppressor devices.

Summary

The investigation may be summarized as follows:

1. Laundries and certain large water-using industries frequently employ quick-opening and -closing valves. These use all types of power operating devices in addition to manual operation. They can and do create water hammer and surge problems.

2. The water hammer disturbances evidence themselves on the customers' premises more generally in noise and rattling pipes, and sometimes in ruptured valves or appurtenances.

3. The repetitive nature of the operations in the equipment in which these valves are used makes it desirable and necessary that the water customer apply corrective measures.

4. Air chambers, snubbers offered under various trade names, adjustable rates on quick-operating valves, and even hose connections have been used to reduce the effect of, or to eliminate, water hammer and its associated problems.

5. Among many of the laundry equipment manufacturers contacted, there seemed to be little understanding of the principles of water hammer. It could be inferred that the initiative for correction was left to the user.

6. One of the large industries emphasized that it was its responsibility to see that disturbances on its premises did not reach the mains.

The entire inquiry seems to justify a conclusion that surges on customer premises due to repetitive operation of valves on process equipment will probably not be serious in the mains of the water purveyor, because the customer will find that dangerous pressure conditions, noise, or rattling of pipes will have to be corrected to assure trouble-free operation of his water system.

APPENDIX 3

Statements of Industry Representatives

Statements from representatives of the four pipe-manufacturing industries were included in the 1952 panel discussion on water hammer allowances. Each industry representative on this committee was asked to review the 1952 opinions and prepare a new statement for this report. These statements represent the opinions of the pipe manufacturer representatives on allowances for water hammer.

Cast-Iron Pipe

Arthur E. Schuh, Director of Research, US Pipe & Foundry Co., Burlington, N.J.

The member of the committee on water hammer allowance representing the cast-iron pipe industry was asked to look into the following questions:

1. Does the water hammer allowance shown in current cast-iron pipe standards apply to modern cast-iron pipe, realizing that these allowances were originally assumed to be adequate for pit-cast cast-iron pipe?

2. Is a water hammer allowance necessary or advisable in the current cast-iron pressure pipe standards?

Before these two questions are answered, it is necessary to point out that the committee on water hammer allowance has limited its considerations to water distribution lines up to a maximum of 18-in. diameter, and that pump discharge mains, transmission mains, and pipe installations other than distribution systems are excluded. This limitation greatly mitigates performance hazards that might be attributable to water hammer.

Applicability of Allowances

The values on which the calculations of cast-iron pipe properties are based for the old pit-cast product and the modern centrifugally cast pipe are as follows: for pit-cast pipe, modulus of rupture is 31,000 psi and bursting tensile strength is 11,000 psi; for centrifugally cast pipe, modulus of rupture is 40,000 psi and bursting tensile strength is 18,000 psi. Experience over many years has shown that the mechanical property values for centrifugally cast pipe are well on the conservative side.

On the basis of these values, Table 2 shows certain calculated minimum bursting strengths over the pipe diameter range from 4 to 18 in., for both pit-cast and centrifugally cast pipe. The wall thickness shown in the table represents net wall thickness, in which the corrosion and foundry tolerances were subtracted from the standard thickness tables. These tables, based on the use of Class 150 pipe with Type B laying conditions and 5 ft of cover, include a factor of safety of 2.5 applied to both the static-design pressure and water hammer allowance. The bursting pressures shown are calculated for a condition without cover.

Table 2 also shows the number of times that the calculated bursting pressure exceeds the assumed-use pressure of 150 psi. It will be noted that though the net thickness of centrifu-

TABLE 2

*Calculated Minimum Bursting Strengths for Class 150 Pipe**

Pipe Size *in.*	Pit-Cast			Centrifugally Cast		
	Net Wall Thickness *in.*	Bursting Strength *psi*	Times Working Pressure*	Net Wall Thickness *in.*	Bursting Strength *psi*	Times Working Pressure*
4	0.25	1,375	9.2	0.22	1,930	12.3
6	0.28	1,020	6.8	0.25	1,750	11.7
8	0.31	840	5.6	0.28	1,230	8.2
10	0.38	830	5.5	0.30	1,050	7.0
12	0.42	765	5.1	0.34	1,000	6.7
14	0.47	720	4.8	0.35	860	5.7
16	0.52	695	4.6	0.38	820	5.5
18	0.63	755	5.0	0.42	800	5.3

* Working pressure of 150 psi.

gally cast pipe is less than that of pit-cast pipe, the number of times by which the calculated bursting pressure is in excess of the operating pressure is greater for the centrifugally cast than for the pit-cast pipe. A generous margin for water hammer allowance is indicated.

Since Table 2 is based on calculated bursting strengths, it was considered significant to examine the record for actual bursting values of modern centrifugally cast pipe. For this purpose, the records of recent bursting tests in the 6-in., 8-in., 12-in., and 16-in. sizes are used. These tests were made on pipe manufactured by the writer's company, and are not cited as representing industrywide sampling. The results are shown in Table 3 and in Fig. 1.

A comparison of Tables 2 and 3, which in effect compare design to performance, shows that the average actual bursting strength is considerably higher than the design strength and the individual bursting-test values appearing in Fig. 1. All exceed the calculated values of Table 2. This condition reflects the fact that, for manufacturing considerations, cast-iron pipe has a wall thickness greater than is called for in the design.

Adequacy of Current Standards

Sections 1–7 of the Standard for the Computation of Strength and Thickness of Cast Iron Pipe AWWA C101 (ASA A21.1) concern water pressure and water hammer (*1*). Included there is a brief table, based on assumptions for water hammer, which over the size range under consideration varies from 100 to 120 psi. These allowances have been used in establishing the thickness for centrifugally cast pipe as given in Standards AWWA C106 (ASA A21.6), Tables 6.3 and 6.4, and AWWA C108 (ASA A21.8), Tables

TABLE 3

*Actual Bursting Strengths of Class 150 Pipe**

Pipe Size *in.*	No. of Specimens	Avg. Min. Wall Thickness *in.*	Avg. Actual Bursting Pressure *psi*	Times Working Pressure*
6	50	0.35	3,087	20.6
8	50	0.39	2,485	16.6
12	50	0.47	1,977	13.2
16	30	0.56	1,687	11.2

* 150-psi working pressure.

8.3 and 8.4 (2–3). Although the allowances included in the design of pipe listed in these tables are considered ample for most installations, AWWA C101, Sections 1–7, includes the following statement, "Each designer of a pipeline should consider whether the conditions in his case may need a more liberal water hammer allowance."

Since a rigorous analysis of the probable magnitude of water hammer for a given installation is not as yet feasible for distribution systems, and hence such allowances are not quantitatively predictable, it is pertinent to examine the record of performance of pipe whose thickness was predicated on the inclusion of the above allowance.

Other members of the committee have been examining the long-time performance record of cast-iron pipe in several major cities. The evidence to date, as it pertains to normal operating conditions in a distribution system, has indicated no substantial number of bursting failures ascribable to water hammer. This affirms the adequacy of design with respect to water hammer allowance.

There are, however, special instances in which excessive water hammer can be developed, sufficient to cause bursting failure in a small-diameter distribution line. These instances are usually associated with automatic quick-acting valves. As Vance C. Lischer points out, the frequency of occurence of such a condition is low and usually isolated, it is readily recognized, and it can also be anticipated. Adequate and straight-forward corrective measures are available. It is certainly to the interest of the water distribution agencies to implement a systematic preventive and corrective policy.

In the meantime the question of what to do about this restricted area of water hammer caused by quick-acting valves, appears to the author to be primarily one of judgment. In those instances where a new installation is expected to cope with the presence of quick-acting valves, it may be simplest to provide a line with pipe of heavier wall. In other instances where a change to quick-acting valves is introduced after the distribution piping is already in place, the provision of suitable surge-controlling devices should be insisted upon.

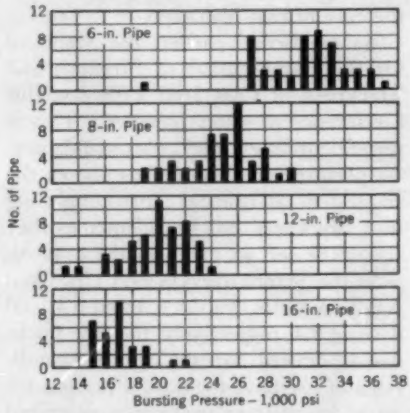


Fig. 1. Results of Bursting Tests on Cast-Iron Pipe

Tests were made on specimens of Class 150 pipe, designed for working pressure of 150 psi. There were 50 specimens each of 6-in., 8-in., and 12-in. pipe, and 30 specimens of 16-in. pipe. The average bursting pressure for each group is shown in Table 3.

In the opinion of the author there is no need for a change in the stipulated allowance in the cast-iron pipe specifications for water hammer in distribution piping within the size range in question, for the following reasons:

1. Cast-iron pressure pipe has in its design, and as it is being manufac-

tured, an adequate margin of safety against bursting stresses to cope with all normal operating conditions.

2. In those instances where very quick-acting valves enter the picture, available corrective steps can and should be utilized.

If the above can be accepted as reasonable, there still remains the question of what it may be advisable to do by way of standards, particularly in the case of cast-iron pipe.

As mentioned earlier, the Standard for the Computation of Strength and Thickness of Cast Iron Pipe contains a reference to water hammer. This is purely advisory and not mandatory. As performance history has shown the suggested allowances for computing wall thickness to be adequate, they should be left as given in the manual.

In the Standards AWWA C106 and AWWA C108, Tables 6.3 and 6.4, and 8.3 and 8.4, respectively, the pipe thicknesses offered for the various conditions of use have included a margin for water hammer as previously described and should also be left undisturbed. There is a need, however, for clarification of footnotes accompanying these tables in that they might well show what actual water hammer allowances were assumed in establishing the wall thickness.

References

1. American Standard for the Computation of Strength and Thickness of Cast-Iron Pipe—AWWA C101 (ASA A21.1). Am. Wtr. Wks. Assn., New York.
2. American Standard for Cast-Iron Pipe Centrifugally Cast in Metal Molds, for Water or Other Liquids—AWWA C106 (ASA A21.6). Am. Wtr. Wks. Assn., New York.
3. American Standard for Cast-Iron Pipe Centrifugally Cast in Sand-Lined Molds, for Water or Other Liquids—AWWA C108 (ASA A21.8). Am. Wtr. Wks. Assn., New York.

Asbestos-Cement Pipe

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To date, the information available does not appear to justify changing the basic method of making allowance for water hammer as set forth in the panel discussion presented in 1952 at the Annual Conference in Kansas City (*1*).

Essentially, this method provides for a relatively large factor of safety for hydrostatic strength so as to provide for the many variables such as water hammer, field test pressures, backfill loads, surface loads, impact, and internal and external corrosion encountered in distribution systems. The original factor of safety followed the European practice of requiring twice the recommended working pressure. This was changed to four times the recommended working pressure in the year 1936 to provide the degree of performance required in the United States.

The author feels that if an allowance is made for water hammer in the design of pipe, similar allowance should also be established for all the other variables which affect the ability of the pipe to withstand hydrostatic pressure. It would be impractical to establish such levels and designate those combinations in which they should be used.

Today, practically all asbestos-cement pressure pipe used in municipal distribution systems complies with the AWWA Tentative Standard C400 for Asbestos-Cement Water Pipe (*2*), and as far as the author knows, such pipe has satisfactorily withstood the degree of water hammer normally encountered in such systems.

The design of asbestos-cement pipe should be based on the requirements of

a normal distribution system, and such a design should not be penalized by the rare abnormal condition occasionally encountered.

References

1. KERR, S. L., ET AL. Standard Allowances for Water Hammer. *Jour. AWWA,* **44**:977 (Nov. 1952).
2. Tentative AWWA Standard for Asbestos-Cement Water Pipe—C400. Am. Wtr. Wks. Assn., New York.

hammer pressure exceeds 50 per cent of the normal operating pressure, there is no danger whatsoever of rupturing a steel water pipe. A further safety factor also exists in steel pipe installations of 18-in. diameter and smaller sizes in that the thickness generally used is greater than required for normal operating pressure alone. For example, the latest AWWA design factor for 18-in. diameter, 0.135-in. thickness

TABLE 4

*Recommended Design Factors for Steel Pipe**

Outside Diameter *in.*	Min. Wall Thickness *in.*	Safe Working Pressure† *psi*	Bursting Strength *psi*	Times Working Pressure	Times Working Pressure Based on 150-psi Service
4½	0.105	700	2,427	3.5	16
6⅝	0.105	476	1,650	3.5	11
8⅝	0.105	365	1,265	3.5	8.5
10¾	0.135	376	1,303	3.5	8.7
12¾	0.135	318	1,102	3.5	7.4
14	0.135	289	1,002	3.5	6.7
16	0.135	253	877	3.5	5.9
18	0.135	225	780	3.5	5.2

* Based on AWWA C202 (*1*).
† Based on a 15,000-psi stress.

Steel Pipe

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For water distribution systems only, using pipe sizes of 18-in. diameter and less, AWWA Standard C202 (*1*) represents good design practice for steel water pipe safely to resist static and water hammer pressures that may occur under these conditions. According to the standard, normal working pressure should be 15,000 psi stress; working pressure plus water hammer allowance should be 22,500 psi stress (150 per cent working pressure); and hydrostatic test pressure should be 25,500 psi stress. Unless the water steel pipe, which is the minimum thickness permitted for this diameter, will give a safe working pressure of 225 psi, a hydrostatic test pressure of 380 psi, and a bursting pressure of 780 psi. There are very few distribution systems that would ever approach these values.

Data in Table 4 indicate the generous safety factor that exists for steel pipe under the AWWA standard to guard against water hammer in distribution systems for sizes of 18-in. diameter and less.

Reference

1. AWWA Standard for Steel Water Pipe of Sizes up to But Not Including 30 Inches—AWWA C202. Am. Wtr. Wks. Assn., New York.

Reinforced Concrete Pipe —

Hugh F. Kennison, Vice-Pres., Eng. and Research, Lock Joint Pipe Co., East Orange, N.J.

Since the November 1952 JOURNAL publication on standard allowances for water hammer (*1*), there have been changes in Standards C300, C301, and C302 (*2–4*). These revisions warrant an amended statement of controlling factors. Standard C301 has now incorporated prestressed concrete embedded cylinder pipe. Standard C302 has allowed higher theoretical steel stresses for calculating the operating pressure.

Based on the revised standards and an additional extensive test program, Table 5 presents controlling design criteria for concrete pressure pipe. These factors represent conservative inherent properties of pipe for resisting static and water hammer pressures. For consistency of expression all factors are expressed in terms of the working pressure P_w. References to the zero compression pressure for prestressed pipe have been deleted.

TABLE 5
Design Criteria for Reinforced Concrete Pressure Pipe

Pressure	Steel Stress *psi*
Steel Cylinder Type, Not Prestressed*	
Design pressure, P_w	12,500
Static head (line gate valves closed), 1.2 P_w	15,000
Design pressure plus water hammer, 1.4 P_w	17,500
Noncylinder Type, Not Prestressed†	
Design pressure, P_w	12,500
Static head (line gate valves closed), 1.1 P_w	13,750
Design pressure plus water hammer, 1.2 P_w	15,000

Pressure	Table 3a‡§ (16–48 in.)	Table 3a‡‖ (54–72 in.)	Table 3b‡‖ (24–72 in.)
Steel Cylinder Type, Prestressed‡			
Design pressure	P_w	P_w	P_w
Static head (line gate valves closed)	1.25 P_w	1.35 P_w	1.5 P_w
Design pressure plus water hammer#	1.4 P_w	1.6 P_w	1.75 P_w

* AWWA C300 (*2*).
† AWWA C302 (*4*).
‡ AWWA C301 (*3*).
§ Both lined-cylinder and embedded-cylinder types.
‖ Embedded cylinder.
\# All designs have at least 50-psi allowance for water hammer.

References

1. KERR, S. L., ET AL. Standard Allowances for Water Hammer. *Jour. AWWA*, **44**:977 (Nov. 1952).
2. AWWA Standard for Reinforced Concrete Water Pipe—Steel Cylinder Type, Not Prestressed—AWWA C300. Am. Wtr. Wks. Assn., New York.
3. AWWA Standard for Reinforced Concrete Water Pipe—Steel Cylinder Type, Prestressed—AWWA C301. Am. Wtr. Wks. Assn., New York.
4. AWWA Standard for Reinforced Concrete Water Pipe—Noncylinder Type, Prestressed—AWWA C302. Am. Wtr. Wks. Assn., New York.

Study of Breaks in Water Mains at Louisville, Ky.

Byron E. Payne

A paper presented on May 13, 1957, at the Annual Conference, Atlantic City, N.J., by Byron E. Payne, Chief Engr. & Supt., Louisville Water Co., Louisville, Ky.

A STUDY has been made of the breaks in cast-iron mains in the Louisville Water Co. pipe systems for 1941–55. Breaks were classified according to pipe sizes, months, seasons, types of joint material, and years of pipe installation.

Each month a report of the breaks was submitted, giving the following information for each one: date of break, location, size of pipe, depth of cover, type of break (circumferential or longitudinal), kind of pipe, kind of joint, and year installed. These data were tabulated in various ways, as follows:

1. Breaks by sizes for each month, season, year and totals for the period
2. All breaks by months and seasons
3. Breaks by type of joint materials used for months, seasons, and years
4. Breaks by years installed for each month, season, and year
5. Breaks by depth of cover and geographic location for months, seasons, and years.

Results of Study

In each case, comparisons were made, taking into account the number of breaks, average per year, percentage of total breaks, and size of mains. Averages were based on a standard of 100 miles of pipe.

Ninety-three per cent of the breaks occurred in 6-in., 8-in., and 12-in. mains. For each 100 miles of main, breaks were distributed on the average as follows: 13.2 per year for 6-in.; 16 per year for 8-in.; 12.6 per year for 12-in. mains; and 0.6 per year for all other sizes.

Seventy-six per cent of breaks in all cases occurred during the winter months—from November through February—an average per year of 93 in the winter, compared to 30 during the rest of the year. Figures for each 100 miles of pipe averaged 11.3 per year in the winter and 3.7 per year during the other 8 months. A study of the comparative effects of sulfur jointing compounds and lead joints showed that 78 per cent of these winter month breaks were in pipelines using the sulfur jointing compound as the calking material.

A study was made based on type of pipe used, period installed, percentage of breaks, and averages per year. Results are shown in Table 1. In this study, the depth, type of pipe, and geographical location did not point to any conclusion. Also, the type of break was indicated to be a function of the kind of pipe.

The Louisville Water Co. used pit-cast pipe Class C up to 1925 and centrifugally-cast pipe Class 150 since.* It used lead joints up to 1922, sulfur

* DeLavaud (horizontally centrifugally cast) pipe.

TABLE 1

Data on Main Breaks in Louisville, Ky., 1941–55

Period Installed	Type of Pipe	Total Breaks		Average Breaks Per Year	
		per cent	*per 100 miles*	November–February *per 100 miles*	March–October *per 100 miles*
Up to 1925	Pit-cast	24	48.5	3.3	2.1
1925–30	Centrifugally cast	59	310.0	29.0	5.0
1930–52	Centrifugally cast	17	96.0	7.6	3.2

joints from 1922 to 1954, and mechanical joints since 1954. Water temperatures in the mains range from 85°F in the summer to 34°F in extreme winters.

Very few breaks can be attributed to water hammer. Over a long period, the system recording pressure gage has been checked for evidences of pressure surges immediately preceding a siege of main breaks; very few such occurrences have been observed. Some meter frost bottom blowouts following pressure surges, however, have been noticed.

Following the inundation during the 1937 flood, when about 50 per cent of the area served by Louisville's system was under water, many breaks and leaks were anticipated because of the inundation and compaction of the flooded underlying sand and gravel strata, broken sewer connections, catch basin lead failures, washouts, and cavities under the street surfaces. But during this 30-day flood period, there were only 25 breaks in 4-in. through 12-in. mains. A total of 215 breaks and leaks in the distribution system occurred during the year 1937, compared with 203 reported during 1934; 198 during 1933; 170 during 1938; 156 during 1939; and 240 during 1940. This information would indicate that the flood had little effect on breaks and leaks in Louisville.

Conclusions

In analyzing the causes of breaks in cast-iron water mains, it appears that most breaks are the result of the culmination of a number of possible conditions. The above data and other observations indicate that these conditions tend to appear in the following order of frequency:

1. Water temperature stress
2. Rigidity of joints
3. Certain pipe material failures
4. Ground movement
5. Ground disturbance by sewer construction
6. Faulty bedding during original construction of rock trenches
7. Water hammer stresses.

Role of the Attorney in Multipurpose Developments

Porter A. Towner

A paper presented on Oct. 30, 1957, at the California Section Meeting, San Jose, Calif., by Porter A. Towner, Chief Counsel, Dept. of Water Resources, Sacramento, Calif.

ANY utility executive can imagine himself standing in a courtroom, which is nearly empty. Only one reporter from a local paper is there to cover the case. Besides the parties involved, a few curious people have wandered in. The trial has been long and tedious, and public interest has waned, although the case is of vital importance to the utility. The executive notices that his face is perspiring as he stands. The judge comes in, ready to hand down his decision, the decision that spells life or death to a dam, the dam so vitally needed to keep water projects going.

The prevention of a threat such as this is one of the principal functions of the utility's lawyer. He is sure to advise that every precaution should be taken at the earliest possible stage of any project to avoid potential legal conflicts, and, if such conflicts are unavoidable, that settlement through a fair agreement is always better than litigation, for litigation is costly and time-consuming. Water litigation vies with that in the antitrust field for the dubious distinction of being the most costly and the most time-consuming. At the height of one celebrated recent case (*1*), the cost to the parties was estimated at $4,000 per day, the value of more than 1,000 acre-ft of water from the project involved. Another example is a case that has been in progress for more than 30 years (*2*). The expense of court proceedings will, of course, increase the cost of the project. Even worse, the new project may be delayed for years awaiting the final outcome of the lawsuit.

Of course, not every lawsuit can be prevented; it takes two to agree. There may be irreconcilable differences of opinion or questions concerning which the law is unclear or has not yet been determined. It goes without saying that if litigation ensues, the lawyer's job is to win it if he can. The preparations to avoid litigation, or to win the law suits that do come, must commence far in advance.

The mechanics of multipurpose water development are becoming of vital interest to water utility management, since it is increasingly to this source that municipalities must look for new water supplies. The multipurpose project may be one constructed by a city providing for power production, flood protection, or irrigation water, as well as municipal water supplies, or it may be constructed by the state or federal government. It may be a cooperative project involving other local agencies or the state and federal governments. At any rate,

major single-purpose municipal water development, at least in the West, is almost a thing of the past.

In carrying out a multipurpose water development, the attorney must work as part of a smoothly integrated organization. He must correlate his efforts with administrators, engineers, and those with other specialized abilities. Particularly in water matters, law, engineering, and economics are likely to be inextricably intertwined.

Planning the Project

The lawyer's work should begin in the earliest phases of the planning of the project. He must try to anticipate the legal problems that will arise. He must work closely with the engineering staff and furnish it with the information with which it can plan a project that will avoid serious or insoluble legal problems.

One of his first tasks in this connection is to ferret out the law that is applicable to the proposed project, both legislative enactments and judicial precedents. Unfortunately, in the multipurpose project field, this is none too easy, for many of these projects are in areas where the law has not yet been defined. He and the engineers working together must plan the project to avoid as many legal pitfalls as possible. Because some of the laws may be outmoded, having been designed for projects of an early day, new legislation may be needed, and it must be planned far in advance. All this often must be done under the pressure of events where full reflection and discussion is not possible, for often water needs will not wait.

One of the most important phases in the planning of the project is the handling of water rights. In California, applications must be filed and advertised, opportunity must be given for protests, and, usually, a hearing must be held before a permit for the use of water can be obtained. This, too, requires very close coordination with the project planners, for there is a considerable time lag between application and permit. The applications must be filed at the earliest possible date at which the information needed for them is available, and the information needed must be developed in the planning at an early stage. It may also be desirable to purchase water rights, and it is always necessary to compensate the owners in case of damage to water rights. These objectives may also be accomplished by an exchange of water or other physical solution of the problem.

The California area of origin laws present a good example—one of particular concern to the Department of Water Resources—of laws with which a project must be made to dovetail. Leaving aside the question of change in the law, a state project must be planned so as not to deprive areas or counties of origin of water rights guaranteed to them by the legislature. Whether or not a project meets these conditions is both a legal and an engineering question. If the present law does not furnish adequate guarantees to those in the areas of origin or provide firm water supplies to water users in the export areas, legislative or constitutional changes must be considered. The lawyer plays a central part in implementing policy through interpretation and drafting in the preparation of new legislation. Careful work in preparing legislation will make it reflect clearly the desired policies and will avoid much future controversy.

If the project involves power features—and many municipal water sup-

ply projects utilize power revenues to defray part of the cost—an application to the Federal Power Commission will be required. Here again, timing and coordination of efforts are crucial. The application may result in an administrative hearing or even in court action, if there is competition for the power site or if there is organized opposition to the planned project.

Another phase of the work in which the utility attorney should take the lead is in keeping track of developments that may affect planned projects. In particular, legislation and court decisions require careful scrutiny. While the legislation is pending and before the court has decided, it may be possible to make representations to the legislative body or the court to the end that a decision will be made that will help and not hinder a proposed project. If the issue has been unfavorably determined, curative measures can often be taken. Under any circumstances, the lawyer must interpret the results of judicial or legislative pronouncements so that their effect on the project, not always at once apparent, can be readily understood.

An example of the kind of vigilance that is required was furnished by the Pelton Dam case (*3*). When the Supreme Court decided in that case that the builder of a power project, using withdrawn and reserved federal land, need not comply with the state water law, lawyers working on municipal and other water developments immediately began to evaluate its implications. Although it involved a power project in Oregon, this decision may well have profound effects on water developments in California. Viewed in the perspective of western water law and prior decisions of the Supreme Court, it threatens substitution of federal planning for state and local planning and raises doubts as to the basis of all vested water rights. By way of corrective measures, legislation is now pending in Congress which would reaffirm the validity of state water law and require federal compliance with it. The best known of these measures (*4*) is the Barrett Bill, so named for its author, Senator Barrett of Wyoming.

Authorization and Financing

All water projects constructed by public agencies must be authorized by affirmative legislative action. Depending on the agency, this may involve action by the legislature, a resolution by the board or council, or a vote of the electorate. At any rate, the authorizing measure must be given the most careful consideration, since it will govern the future conduct of the project, and its amendment may be difficult. It must contain authority for the exercise of the powers needed to construct and operate the project and be free from limitations that will hamstring it or unjustifiably increase the cost. In addition to primary responsibility for drafting, in which process the lawyer must consider every possible interpretation that may be given to the language, based on his knowledge of statutory construction, the lawyer must also function as an interpreter. Because of the surprising ways that courts have construed seemingly plain language, legislative measures must contain safeguards and may carry implications that are not readily apparent. It is the lawyer's job to see that the administrator and the engineer understand the effect of proposed authorizing legislation on the conduct of the project, and to see that the legislation that is framed carries out as nearly as possible their intent.

Closely related to the problem of authorization is the financing of the project, for it goes without saying that the authority is empty unless the funds to do the work can be obtained. Few indeed are the projects that can be financed solely from current funds. Most multipurpose projects will require issuance of bonds, the procuring of a loan or grant, or both.

The issuance of bonds, besides the financial consideration, is filled with legal requirements which must be met with precision. The lawyer is likely to call in a consultant who specializes in the legal aspects of securities, unless the organization is big enough to have such a specialist on its staff. The specialist will give expert legal consultation, much as a specialist in one branch of medicine might consult with a family doctor. Unless the project is financed by revenue bonds which are serviced with revenues from the sale of water or power, an election will ordinarily be required. Again, careful attention must be given to meeting legal requirements designed to insure the fairness and regularity of the election. For certain political subdivisions of the state, approval of the bonds will have to be obtained from the California Districts Securities Commission, and a court validation may be required or may be advisable. The lawyer generally handles these technical matters with coordination as to timing, and with consultation if policy questions should arise.

Many times financial assistance or cooperation may be obtained from other governmental agencies. The federal government has a well-established policy of participating in multipurpose projects to the extent of flood control benefits. Also, the Small Reclamation Projects Act of 1956 (*5*) authorizes federal participation to the extent of $5,000,000 in certain types of projects, primarily for irrigation, which may, however, include municipal water service. It may also be desirable to cooperate with other state or local agencies to build a single project that will meet the collective needs better than several smaller projects. The Cherry Valley Project of the city of San Francisco exemplifies the possibilities of this approach, for the city cooperated in that project with the Corps of Engineers and the Modesto and Turlock irrigation districts.

Recently a new factor has emerged through a law adopted by the 1957 California legislature, under which the state may cooperate in the financing and construction of water projects. Chapter 2052 of the California Statutes of 1957 establishes policies for state participation. Subject to specific legislative approval, it contemplates loans to public agencies or grants for fish and wildlife and recreational purposes incidental to the project's main purposes, with the added requirement that recreational benefits must be of statewide interest. State participation is also authorized to provide for full development where that is beyond the requirements of the project builder. Under this law, assistance will be available for municipal water development.

The lawyer, economist, and engineer must work together in weighing the many possibilities for financial assistance or cooperation to determine what plan will best meet the particular situation. Additional legislation will often be required; for example, in most projects congressional action is necessary for federal flood control participation, and the legislature must act to approve state participation. For any financial assistance, a contract will generally be

required to spell out the rights and obligations of each party.

Construction

When the project is authorized, and its financing is assured, the utility apparently is ready to start with construction. There remain some legal problems, however, which must be solved first. The land for the project must be acquired. Roads and utilities must be relocated. If this work is not to delay construction, it must be initiated far in advance of the letting of the first construction contract.

If it is the first construction project of the agency, forms and procedures will have to be established for the acquisition of land. Since most of the problems will involve meeting the requirements of law, the lawyer can take the lead in solving these problems. Later, he will probably review the transactions to be sure that the law has been followed and the necessary title obtained. Where agreement with landowners cannot be reached or construction schedules do not allow time for negotiations, condemnation may be necessary. As with all litigation, condemnation is likely to increase expenses and should be used only when really necessary. Even after the institution of eminent domain proceedings, efforts should be continued to settle the case before trial. Large scale relocation contracts are likely to require even more time, and negotiations should be commenced as early as possible.

With respect to construction contracts, there will be required careful legal review of plans and specifications and bid and performance bonds to assure compliance with the applicable laws and to avoid future contract interpretation difficulties. If matters of contract interpretation do arise, the attorney must assist in determining such matters as the issuance of extra work or change orders and the assessment of liquidated damages. During the construction phase of the project, the attorney's role is, in general, one of watchfulness to anticipate legal difficulties and assure compliance with legal requirements. He must rely on the construction engineers to keep in close touch with him concerning situations which may be legally hazardous.

Operating the Project

Unless the agency that has constructed the project utilizes its total output of water and power in supplying ultimate consumers, contracts for the furnishing of these commodities to other agencies will be required. Also, if the project is a cooperative one, contracts will be necessary to establish operating criteria for each agency. Work on these agreeements will have to be commenced during or before construction if they are to be consummated by the time the project is ready to go.

With the contracts made and the project operating, legal problems are still not at an end. With proper planning and consultation, they can be reduced to a minimum, but the variety of problems that can arise is infinite. Proper consultation and planning include the furnishing by the attorney of information to the operators as to legal requirements for the avoidance of injury to others. It is also necessary to have discussions between the operators and their attorney concerning actions that might involve liability before rather than after they are taken.

Some claims of legal liability are inevitable, however, for both lawyers and operators are human. Accurate records, prompt investigation, full reports,

and efficient handling will tend to keep liability to a minimum and will promote just settlements.

Litigation

Litigation may arise in many phases of the work. It may develop from the applications for water rights or for a power license, from opposition to authorization, in acquiring land, or because of a disgruntled contractor. It may arise any time that a disagreement as to the interpretation of one of the many contracts involved is not settled amicably. Although it should be avoided if possible, it is not the ultimate disaster and actually plays a constructive part in the overall effort. The American judicial system is the best instrument that has yet been developed for settling conflicts between people. It is far from perfect, but it is more likely to be just than a resort to judicial decisions, based on political influence, by the legislative body. It is the legal system which is principally responsible for the rarity with which force and violence are now used to settle disputes and it should be remembered that only two or three generations ago these ultimate arbiters were far from uncommon.

In the court proceeding and in the preparation for trial, the lawyer must take charge. To make its best showing, the utility should give him its fullest cooperation, but this is his field—the responsibilities and legal decisions are his.

Conclusion

The attorney's role will vary considerably during the various stages of planning, construction, and operation of a multipurpose water development project. Throughout, however, it should be characterized by close coordination and teamwork with the utility. The other members of the team are the other specialists in the organization. By developing mutual understanding of the part each specialist plays, and by mutual reliance on each other for work within the particular specialty of each, the organization will be able to carry out the project with maximum efficiency.

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Responsibilities, Authority, and Prerogatives of the Water Utility Manager

Wendell R. LaDue

A paper presented on Jun. 12, 1957, at the Pennsylvania Section Meeting, Bedford, Pa., by Wendell R. LaDue, Chief Engr. & Supt., Bureau of Water & Sewerage, Akron, Ohio.

IN these times of stress, the great need of the people of this country will be for trained and fearless leadership, free to provide the citizens with an intelligent and economical management of those activities which have become generally accepted as normal and routine. In no place will this be more essential than in the utilities field, and particularly in water supply management.

The need for a study of management and the determination of good practice in this phase of water utility endeavor has been very forcibly brought to the attention of the members of the AWWA in the past few years. Although perhaps not designated as such, there is probably no topic which waterworks men discuss more frequently than management, projected under such subjects as: continuity of employment, control of funds, consumer relationship (public relations), adequate plant planning, enabling legislation, safety practices, compensation, personnel relations, water use, rate schedules, fire protection, air conditioning, plant rating, pensions, main extensions, financing, joint administration, accounted-for water, and rules and regulations.

Discussion of these topics arises often from the premise that inadequate attention has been given to underlying and contributory factors in their occurrence. In other words, discussions are held on the results of bad management, rather than its causes.

Concepts of Management

Just what is management? The words management and administration are practically synonymous, and they mean to conduct or to carry on the affairs of business. In this case, that business is water utilities.

All in all, water utility management covers a wide but very specialized experience in a field most necessary for human endeavor, involving the handling of men, materials, commodities, and services in the delivery of a product for which there is no known substitute. Good management results in, and is the result of, a logical, sane policy of plant expansion, sound financing, smooth personnel relationships, efficient administration, and satisfying consumer relationship. Good management is as essential to the small village system with a handful of employees as it is to the large organization employing hundreds of men and serving thousands of consumers.

In our thinking about this subject, however, there is often an error—all too common in the past—of thinking of the "big-plant man" and the "small-plant man" as two different kinds of men. In deed, and in fact, they are

not, for the same kind of man is seen in constant and daily need of different approaches to the same kinds of problems. Progress in solving the problems of management of the small plant cannot be made by driving imaginary wedges between large water utilities and small ones, or by assuming that the one is enlightened, well-meaning, and knows all the answers, while the other is not. In fact, there are outstanding examples of excellent management all along the line, from the smallest to the largest utilities.

A pertinent question then is in order: When does a plant cease to be small and therefore become large? A ready answer can be obtained only by inference. As of today there are over 125,000,000 people in organized communities in this country who receive safe and dependable water supply from publicly and privately owned water utilities. On the average this service is supplied by one employee for each 1,250 people served. In other words, about 100,000 workers serve 125,000,000 customers. This is big business, whether a community is served by five or 5,000.

More than 100,000 persons, each possessing an almost infinite variety of skills, training, and assortment of talents, day in and day out, every day of the year without ceasing, are contributing unselfishly and selflessly of their energy, their ability, and their ingenuity to the biggest business (in terms of tonnage) in America. There is no precise or scientific way to measure the achievements or contributions of each, nor is it really necessary to know. It is quite necessary, however, for utility managers to maintain a brave and sympathetic understanding of the approach to mutual responsibilities, prerogatives, and problems, in order to face conscientiously the duties of their respective obligations. In all, they can then expect encouragement from the public they serve and a wider, fuller, and sympathetic understanding of the utility's purposes.

Responsibility

There probably is no truly impartial viewpoint on such an all-inclusive topic as the manager's responsibility. Many books have been and will continue to be written on success in management, and in all the predominating factors are personal—therefore susceptible to training control. While discussing the manager's responsibilities, authorities, and prerogatives, it would be well to consider the broader meanings of these terms.

The phrase is often heard—"With privilege comes responsibility"—but what is the manager's personal relationship to this phase of the Golden Rule? A privilege is a prerogative and implies precedence by virtue of office; a right attached to an office; a superior right to exercise powers; a priority to the exclusion of others. It is well to remember the source of the prerogative, that it be not considered too personal or licensed. With prerogative follows authority. Authority is legal or rightful power; the right to command or act; power in a particular field; personal power due to opinion or esteem; the influence of character; the power resident in a person to gain or win devotion or allegiance. Here there is a bridging which must take place between the legal rights of the manager and personal influence due to his own earned prestige and esteem. Personal responsibility for one's acts is implied; influence, prestige, and esteem are recognized as being earned.

Influence is the act or power of pro-

ducing an effect without apparent force or direct authority; the influence of suggestions; the power arising from station or character; a power exerted over the minds and acts of others. Prestige implies the power to achieve ascendency over the minds of men by conspicuous excellence in a particular field of endeavor. Here is realized the full force of the value of the individual and the need and admonition for the acceptance of personal responsibility.

On the other hand, responsibility implies both accountability and reliability; a charge for which one is responsible, accountable, or answerable; the ability to answer or respond to conduct and obligations; charged with oversight and management; accountable as being trusted with something valuable. What could be more valuable to a city's continued existence than its water supply? The utility's responsibility is indeed grave and its accountability direct.

"With privilege comes responsibility." Are utility managers worthy of the privileges and responsibilities placed in their trust?

Not long ago, the author was asked quite pointedly: "Are you as a water utility man looking at this problem of water utility management (especially financing) through too narrow and jealous an eye?" The implication is obvious, stating that the utility is not set apart and alone, but is definitely part of a business controlled by the people. It must be admitted that the success of any plan of management depends not only on the merits of the organization itself but also upon the continued active support of those who believe in the organization. Many an excellent legal setup for management has failed to produce the results anticipated, due to the fact that those interested in fighting for and obtaining the desired plan failed to give it their continued, active, and whole-hearted support and vigilance after its adoption. Be it good or bad, the people get the kind of management they want and for which they will work. The citizenry forms a decidedly important element in the problem of management. Although sound basic laws under which a water plant must operate are the first and foremost necessity for good water works management, it must be admitted that, somehow or other, poor management can and does exist under ideal basic laws. Most fortunately, the reverse is likewise true, to the eternal praise of those rare individual managers who thrive in adversity and have been endowed with the patience of Job and the wisdom of Solomon. On the other hand, too often efficient leadership and progressive personnel is strangled and retarded by outside agencies, laws, and forces beyond the control of the trained utility manager and his loyal organization.

Qualities of Leadership

Assuming that the manager has accepted his responsibilities seriously, what attributes then will assure his leadership? The American Institute of Management, a distinguished nonprofit organization that rates the management of the country's largest corporations, has devised a check list for determining the ability of top executives. The institute has found that the more successful the executive, the higher he rates on the list, which is based on three items—civic worth, ability and energy:

1. Does he try to live by the Golden Rule?

2. Does he participate actively in community affairs?

3. Does he delegate authority as well as responsibility?

4. Is he respected in his community?

5. In business, does he put first things first and get them done?

6. Does he inspire loyalty and enthusiasm?

7. Has he a good personal credit record?

8. Is he open to new ideas?

9. Is he aggressive and decisive?

10. Is he actively broadening himself through study?

It is seen that the delegation of authority has been considered to be of the utmost importance. A good manager knows when and where to delegate authority, and once done, assumes full responsibility for the results. The success of subordinates depends upon this key axiom of successful organization and managerial ability.

What personal qualities should an individual try to cultivate in order to become a successful manager?

This question was asked of Wheeler Sammons, who has published *Who's Who* for the past 27 years and who has, in that period, studied the backgrounds of millions of successful men and women. Sammons stated that a successful person has developed:

1. Concentration on one goal or idea that is higher than he thinks he can achieve

2. Self analysis, his true desires and then determination, no matter how high the goal

3. Ability to deal with and influence people

4. Capacity for working really hard, for giving all to the job

5. A sense of satisfaction with the job

6. Courage to withstand the reverses that he will often encounter

7. Good physical condition

8. A lively interest outside the job

9. A liberal education that teaches him to think.

On the debit side of the personnel ledger, Sammons listed such items as:

1. The personal necessity for earning a living and supporting a family (Note the word *necessity.*)

2. Conservation or backwardness of his associates in producing a living, vital, forward-looking organization

3. Educational deficiencies in both the manager and his associates

4. Lack of good personnel relations and friction with key men and intimate coworkers (This involves the lack of ability to get along with people.)

5. Personal prejudices inimical to impartial pursuit of his duties and obligations

6. Shortcomings of a personal nature which are negative in effect.

To offset these, a person should aspire to good health, character, ability, teamwork, thrift, vision, and courage.

Manager's Authority

Granted that one is a good manager in terms of the above, how then can he be assured of carrying out his responsibilities to his community? Nationally, there is an expression of confidence in the commission form of management, independent of any regular municipal organization, and private companies usually operate in accordance with, and under, statewide utility rules. In Pennsylvania the authority principle has found favor. The practical fact must not and cannot be neglected or ignored, however, that probably a large majority of municipal water works systems operate, and in all likelihood will continue to operate, under the mayor–council and one-man organization forms.

All too often management is considered to be the more or less routine operation behind the delivery of a sup-

ply of potable water to the consumer's tap. It is far more than that. In a very practical analysis, it must embrace sound planning for the future and the coordination of these future needs, plans, and accomplishments with the financial obligation and ability of the utility and the community to provide and support them. It provides a smooth routine of continuing operation so that when future needs become urgent, adequate planning (both physical and financial) has already foreseen and has made possible the carrying out of those needs with the least possible disturbance to the utility and its customers.

The water utility profession generally deems it essential that the manager of the utility shall have freedom:

1. To develop policy
2. To develop rate structures
3. To control funds for expansion
4. To control personnel
5. To purchase materials and enter into contracts
6. To initiate long-term planning and development.

In addition to these freedoms, a municipally owned water utility generally receives the city's credit in financing, is generally free from some forms of taxation, accepts legal and financial advice, participates in bulk buying by the city's purchasing department, and receives the protection of the fire and police departments. Private companies sometimes enjoy equalizing and offsetting privileges.

While these are termed freedoms, they are nevertheless definite prerogatives and, therefore, are not to be abused by the manager. In fact, in their fulfillment and acceptance rests the appraisal or success of any given manager's efforts. Herein lies the real test of respsonsibility and the real expectation for successful operation.

Manager's Purpose

What is the purpose of successful administration? Good management is the result of a combination of continuing policies with respect to expansion of both facilities and operations, sound financing, efficient conduct of business, and giving satisfying services. Good management is as essential for small plants as it is for large water utilities. This axiom of good management will become mandatory as the public realization of its importance becomes more and more apparent.

The water profession will accomplish much to improve its own status if it can effect an enlightened and progressive top management dedicated to these ideals. The manager should strive continually to analyze the causes that produce weakness and then take steps to correct them by well recognized but generally neglected means of personal effort, backed (as it will be) by an active citizenry.

What is it the average citizen wants and expects from its water supply manager? This inquiry calls for a rather searching and personal form of introspection—often not pleasing to contemplate—but it must be faced courageously and frankly.

The average citizen wants an adequate, safe, sparkling, potable water, at a pressure and in quantity suitable for domestic and industrial needs and, in cooperation with the fire department, sufficient for fire protection. Again, all classes of consumers demand that these functions be provided efficiently and economically, that expansions be made as deemed necessary, and that the rates be reasonable and in step with other items of the cost of living.

Conclusions

In conclusion, it may be said that the water profession will do much to

improve its own position if it can effect a better top management commanding public confidence. Too often, public opinion concludes that the water utility manager is doing a routine job in handling daily occurrences and that a plant, once started, grows and expands under its own momentum. This illusion, then, must give way to a true understanding of the facts, which dictate that good management will result only when continuing policies are allowed to develop and to thrive under merited public approval. The results of such policies definitely are:

1. Adequate facilities
2. Good operation
3. Practical maintenance
4. Reasonable and adequate rates
5. Logical replacement and expansion
6. Sound fiscal methods
7. Satisfying public service
8. Reasonable provision for the future
9. Good organizational procedure
10. Overall efficient and responsible administration.

Present and future needs and problems must be conducive of and met by careful, economic, and sound planning, both as to design and financing. In other words, a sound manager merits public confidence because he cares for the needs of the present and protects, projects, and insures the hopes of the future—a truly honorable undertaking, worthy of the water profession and a challenge to all so engaged.

A good water utility manager recognizes the virtues of his privileges; does not overrate his official authority; values his community influence, prestige, and personal esteem; will not abuse his influence; weighs his prerogatives; and accepts true and evident responsibility as being his accountability to his public for a valuable trust—the operation, maintenance and continuance of that most vital of all utilities, public water supply.

There is no water utility where the manager is obliged to compromise his conscience or to pursue a course which he believes to be socially, morally, or economically wrong, in order to gain the legitimate objectives of the lawful water works enterprise. If this were not so, there would be fewer happy men in the profession. No other career affords a greater sense of inner satisfaction to those who embrace it successfully. In no business is there greater determination shown in trying continually to do a better job.

High Rates of Water Use

Guy R. Scott

A paper presented on Sep. 6, 1957, at the Wisconsin Section Meeting, Milwaukee, Wis., by Guy R. Scott, Prin. Engr., Black & Veatch, Cons. Engrs., Kansas City, Mo.

DURING the past 10 years, there have been many statements and numerous articles describing adequate water service. Some of them advocate that water be made available in the desired amounts, at any time and at any place within the service area, while others favor the more frugal policy of restrictive use. Limitation of supply may require the exercise of restrictive use by some departments not so well favored by nature as others; but those that have a plentiful supply of raw water will find it difficult to rationalize such a policy to a public not fully acquainted with all the problems of collection, treatment, and distribution of water, especially when it is not in complete agreement with the policy.

It should be unnecessary, in sections where water is plentiful, to be forced into the position of defending a restrictive policy. Where water is available for treatment and distribution, it would appear that the water utility profession should be able to make it available, within reasonable service areas, for all reasonable uses. This is a challenge worthy of the ingenuity of the entire industry. The problem of plant design required to provide the service is relatively simple, being primarily one of determining size and number of individual units. Financing such programs, however, is the big problem, because the public, while demanding the service, many times is unwilling to pay the price, at least in the manner by which it is now assessed and collected. A more enlightened public and a less painful method of collection must be developed before a more generous policy can be adopted.

The general growth policy of the profession has always been circumspect, not nearly as adventurous as the pattern followed by the power industry. As a result, the water departments are usually designing and building for imminent load or loads already imposed and requiring service. Perhaps this is desirable; at least there is no excessive backlog of extra capacity for which financing must be obtained. This is the difficult phase of the program—the raising of money for construction, operation, and maintenance. Although many rates are being adjusted to provide for essential enlargements, they have not been developed to the degree that will permit consideration of the more generous policy of water service; and many are yet trying to continue on rates which have been in effect for more than 20 years.

Method of Design

The method generally adopted in the design of a water utility system provides sufficient capacity in the supply

and treatment facilities to deliver water as needed at maximum day demand rates, with high-service pumping and distribution system capacities adequate to deliver water to all sections of the service area at maximum hourly rates of use. Treated-water storage is required in order that these high rates of demand can be satisfied.

In general, where water is plentiful, the intakes and treatment facilities at present prices may cost \$100,000–\$225,000 per million gallons per day capacity. Where water is not plentiful, the acquisition and development of the supply alone may cost more than \$1,000,000 per million gallons per day capacity. The cost of facilities for pumping and distribution of water will vary, depending on layout and construction conditions, from \$50,000 to \$150,000 per million gallons per day carrying capacity.

The most economically constructed and operated system would be one supplying water at a uniform rate throughout the entire year, as all units could then be sized to permit them to operate at full capacity at all times. Such ideal conditions never exist. The activities within the service areas cause the loads to vary from hour to hour within the day and from day to day as the seasons change. The average daily rate is obtained by dividing the annual consumption by 365. The maximum day demand may easily vary from 1.5 to 2 times this average-day requirement, and the maximum hour may reach 3–3.5 times the average daily rate.

Increasing Demands

Living conditions have changed in this period of comparative plenty since the end of World War II. By no means have they become static; they are continuing to change with each new application of science to domestic living. Automobile transportation has made possible the widespread movement from the congested sections of cities to the open suburbs, where lawns and gardens are enjoyed and automatic laundries, dishwashing machines, and garbage grinders are commonplace. These household appliances cause an increase in the average day demand on the system, and the irrigation of lawns and gardens imposes a greater seasonal maximum day and hourly rate than was earlier experienced. If the habits of people were uniform and everyone sprinkled at once, the systems would be unable to supply the demands; the diversity factors are such, however, that the loads may not be more than 2–8 times the average daily requirements. Since the total consumption for irrigation may not be more than 10 per cent of the average annual use, it is evident that plant facilities required to supply this water may be surplus for the remainder of the year. Commonly and with justification, diversity factors have been somewhat improved without hardship by restrictions which permit watering only on alternate days.

In addition to the domestic requirements, there have always been loads due to the commercial and industrial activities. Some of these loads are imposed at a uniform rate, but many of them occur for only a few hours a day. Some are seasonal and for short periods may require water rates 5–10 times their average rate for the remainder of the year.

These maximum rates of use can be illustrated by figuring the costs of facilities for a system with an average capacity of 10 mgd. To figure the cost of providing sufficient capacity in supply and treatment facilities, 10 mgd is

multiplied by 1.75, the average ratio of maximum day demand to average day requirements. The total capacity required, 17.5 mgd, is then multiplied by the average cost per million gallons daily, or $160,000, to give a total of $2,800,000. To figure costs for high-service pumping and distribution facilities, 10 mgd is multiplied by 3.25, the average ratio of maximum hour demand to average daily rate. The total, 32.5 mgd, is multiplied by the average cost per million gallons daily capacity, $100,000, giving a total of $3,200,000. The total costs would then be $6,000,000, or $600,000 per million gallons daily average capacity. Actual experience has shown, however, that these costs may vary from $200,000 per million gallons daily to a figure much higher than the illustrated result.

TABLE 1

Factory Shipments of Self-Containing Air-Conditioning Units

Year	No. of Units	Value $1,000's
1950	257,263	86,913
1951	282,488	93,573
1952	438,464	135,579
1953	1,137,293	293,644
1954	1,436,924	327,369

It is evident that the total cost of a system is definitely affected by the ratios of average to maximum daily and hourly rates. Such variations in demands are accepted as normal, and present rates are intended to make a reasonable recompense for such service.

Air-Conditioning Loads

Within the past 5–10 years there has developed a new seasonal load—that imposed by air-conditioning units using water as a coolant without recirculation. The only differences between this seasonal load and the older seasonal loads are its rapid growth and potentiality for excessive demands if growth is permitted to continue unchecked.

Some conception of the growth of air conditioning may be gained from Table 1, covering factory shipments between 1950 and 1954.

All reports on air conditioning anticipate a continued rapid growth. Forecasters estimate annual retail sales of $2–$5 billion by 1963. If this rate of growth is realized and if the units consist primarily of the nonconserved water-cooled type, as they have in the past, the effect on water utilities will be quite serious.

Precise information on the amount of air conditioning now existing in cities is not generally available. The most recent estimates indicate that in Kansas City the amount is 140 tons per 1,000 population, Chicago 70, Detroit 44, and Milwaukee 60 tons. Of these tonnages, it is estimated that 12 per cent in Kansas City and 85–90 per cent in both Chicago and Milwaukee use water as a coolant with no provision for water conservation. In Milwaukee it was estimated that the water requirement for air conditioning in 1956 was about 61.5 mgd, or about 43.5 per cent of the average daily usage, but of this, only 50 mgd, or about 35 per cent of the average daily rate, was drawn from the city system, as wells supplied the remainder. In Chicago it has been estimated that the rate of usage for air conditioning reached 250 mgd in 1954, or 18.7 per cent of the average-day demand.

The low rates charged for water in Chicago and Milwaukee are no deterrent to the installation and operation of air conditioners of the nonconserved

type, and they can be expected to increase rapidly in numbers in the future unless checked by restrictive measures. Kansas City has an ordinance that prohibits the installation of air-conditioning units using water unless provisions are made for recirculation. With this ordinance in force, the tonnage of such wasteful types is decreasing, as obsolete and worn out units are replaced with new.

TABLE 2

*Average Requirements for Unrestricted Air Conditioning**

Item	Requirements
Design Requirements	
Population	80,000
Air conditioning (100 tons/1,000)	8,000 tons
Temperature of cooling water	55°F
Water requirements—*gpm/ton*	0.8
Operation—*hrs/year*	400
Operation—*hrs/day*	8
Water Requirements	
Rate—*gpm*	6,400
Rate—*mgd*	9.2
Daily requirements—*mgd*	3
Annual requirements—*mil gal*	154
Percentage of annual consumption	4.22%
Additional Costs	
Supply and treatment (3 mgd × $160,000)	$ 480,000
Distribution system (9.2 × $100,000 × 0.56†)	520,000
Total	$1,000,000

* For system designed to produce an average 10 mgd of water, in an area geographically similar to central Wisconsin.

† The 0.56 is based on the assumption that about 50 per cent of the average cost will be required, because most of the demand will be in a restricted area.

The effect of the unrestricted air-conditioning load on the cost of a water system, designed to supply water at an average rate of 10 mgd for a city located in an area geographically similar to central Wisconsin, is shown in Table 2.

The estimated cost of the system to supply water to a city with no appreciable air-conditioning load, having an average demand of 10 mgd, has been shown to be about $6,000,000. If an air-conditioning load such as has been described in Table 2 is then imposed on the original load, the extra cost is estimated at $1,000,000, or a 16.7 per cent increase. The total annual amount of water used for this purpose is not expected to exceed 154 mil gal, or about 4.22 per cent of the average annual use without the air-conditioning load.

Air-Conditioning Charges

The capital cost of the water required for air conditioning is much greater than for the original use, being about $6,500 for the air-conditioning water as against $1,640 for the latter, both in terms of annual sales of water in million gallons. Since it is assumed that the cost of operation and maintenance of the system remains the same per million gallons produced in the two systems, it seems logical that customers having air-conditioning equipment with no provisions for recirculation or any other use that produces low-load-factor conditions should pay a surcharge above regular rates sufficient to carry the capital charges on the investment necessary to provide the service. If such a surcharge is not levied, such service should not be provided, as it should not be subsidized by the remainder of the customers. Such a surcharge is affixed to rates for domestic water to compensate for the low load factor caused by irrigation. The higher unit rates for water used in this category are ample to provide for the capital charges on plant equipment required for such uses.

Annual charges based upon the additional capital cost for air-conditioning water developed in the example are less than the average demand charge for such service. It should be appreciated that this estimate does not represent any particular municipality, as it is the result of applying conservative figures to a set of conditions for illustration only. Representative demand charges presently in use include: Detroit, $7.50 per ton per year; Fargo, N.D., $10; Pine Bluff, Ark., $12.50; Appleton, Wis., $20; and Omaha, Neb., $36 per ton per year. Milwaukee, Wis., is requesting $20 per ton per year.

In 1955, the Missouri Public Service Commission authorized a demand charge of $40 per ton per year for the St. Louis County Water Company, to take effect in May 1957. They have since reversed their decision and have ruled that the cost of this service be spread among all users.

The Missouri Water Co. has special rates for all customers in Independence, Mo., served by 1½-in. and larger meters, whose monthly use during June–September exceeds twice the average monthly consumption for the remaining eight months. These rates are applied to ice plants, bottling works, dairies, creameries, and all customers that have low load factors in these months. They have not been set so high as to handicap such operations, but they are high enough to discourage the installation of air-conditioning units having no provisions for conserving water.

The application of demand charges or special rates for all low load factor uses will not result in a financial burden for reasonable use; they will, however, discourage waste. In the air-

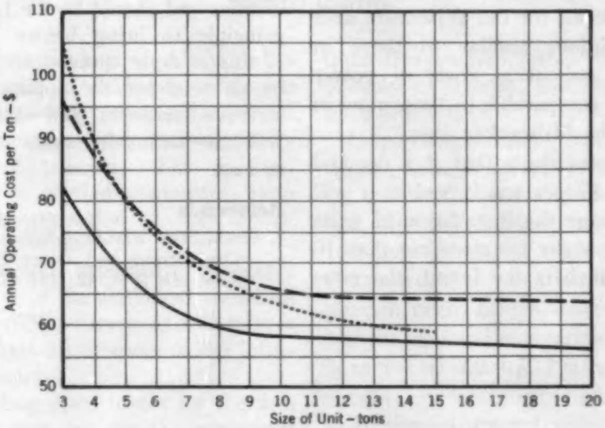

Fig. 1. Typical Annual Costs for Air Conditioning

Total operating costs, including power and water, are shown for air-cooled units (dotted line); water-cooled units that conserve water (dashed line); and water-cooled units that do not conserve water (solid line).

conditioning field the units relying on water as a coolant can be provided with recirculating systems. Such systems will effect a saving of 95 per cent in the use of water. Air-cooled units are also being developed and are now in competition in sizes up to 10 and 15 tons. It is not expected that fair charges for utilities will retard the continued growth of air conditioning.

Annual costs per ton of air conditioning are compared by the curves shown in Fig. 1. These costs have been developed for the Wisconsin area with conditions similar to those in Table 2, and using costs of power and water comparable to those now in effect in the Milwaukee area.

The curves show that if a demand charge of $9 per ton is levied, it will not encourage the installation of units conserving water for sizes less than 10 tons. If such is the intent, the rates should be somewhat larger for the smaller sizes.

Conclusion

All of the articles and discussions on adequate water service heartily support the statement made in the report of the AWWA committee on air conditioning and refrigeration in 1950 (*1*) that "Each customer should bear his share of the burden that he himself creates. . . ." This is the policy that should be adopted and followed by the water utilities and should be the fundamental principle to bring before the public. If this is done successfully, there will be no need for the application of restrictive measures, and each customer will have as much service as he desires to buy.

Reference

1. Committee Report. Regulation of Air Conditioning and Other Refrigeration. *Jour. AWWA*, 42:1111 (Dec. 1950).

Survey on Iowa Water Rates, Main Extensions, and Job Classifications

Iowa Section Committee Report

A report of the Iowa Section Information Survey Committee (Mark E. Driftmier, Chairman), presented on Oct. 17, 1957, at the Iowa Section Meeting, Des Moines, Iowa.

Introduction

THE Iowa Section of AWWA has embarked on a unique service to its members—an annual survey of rates, pay scales, and various utility management policies. This is conducted by a section committee, with the results presented at the annual meeting in October.

This article is intended to present a brief review of the statistical results of the first (1957) survey and to raise a few questions about some of the information disclosed.

In deciding upon topics for the first survey, it was felt by the committee that there were three fields most frequently compared and requiring questionnaires—water rates, the financing of main extensions, and job classifications and wages.

Concerning water treatment and supply, Iowa is served by a wide variety of combinations, from Missouri and Mississippi River sources of 90–150 ppm hardness, which need only filtration and chlorination, to well supplies in north central Iowa which are high in iron and manganese, as well as hardness of 800–1,500 ppm, and which almost defy treatment. Southern Iowa is lacking in ground water, and must depend on surface lakes and streams of varying character and reliability.

Water Rates—*Harris F. Seidel*

A paper presented by Harris F. Seidel, Supt., Water & Sewage Treatment, Ames, Iowa.

To the questionnaire on water rates, 47 Iowa cities and towns responded. They include all cities of 20,000 or more population and a representative sample of smaller municipalities as well. Only three utilities are privately owned; of the public utilities, many are under council or manager control, and a number are under water utility boards. It may be particularly significant to note that Iowa has no regulatory body of any kind to which either private or public utilities are responsible.

Charges

From rate schedules submitted, charges were calculated in terms of

TABLE 1

1	2	3	4	5	6
Community	Date of Last Rate Adjustment*	Rate Steps No.	Rate Steps Range *cents/100 cu ft*†	Quantity Above Which Lowest Rate Applies *cu ft/qtr.*	Billing Period‡
Ames	9/52	3	50 –32	15,000	M
Atlantic	1/55	5	35.2 –12.75	52,000	M, *Q*
Bloomfield	11/52	4	45 –26.25	28,000	M
Boone	1/31*	8	32.5 – 8	100,000	Q
Burlington	6/54	8	30 – 9.75	333,333	Q
Carroll	7/53	7	54 –14.04	12,600	M
Cedar Rapids	1/53*	5	23.75– 9.5	300,000	B
Clarinda	1920*	7	36 –10.8	18,000	Q
Clarion	1951	6	41.25– 8.25	66,667	Q
Clinton	7/55	5	48.75–12.194	400,000	M, *Q*
Council Bluffs	9/52*	4	40 –11	3,000,000	M, *B*
Davenport	2/29	3	30 – 6	399,000	M, *Q*
Decorah	1954	5	36 –13.5	18,000	M, *Q*
Des Moines	12/48	4	30 – 8	2,500,000	Q
Dubuque	6/55	7	36 –10.5	300,000	M, *B*
Eagle Grove	7/54	3	90 –36	2,000	Q
Estherville	1955	4	65 –20	45,000	M
Evansdale	9/56	5	27 –17.25	10,667	Q
Forest City		8	30 – 7.5	53,333	Q
Fort Dodge	11/51*	8	27 – 4.725	133,333	Q
Grinnell	7/54	5	60 –32	5,000	Q
Iowa City	§	4	50 – 9	99,000	M, *B*
Iowa Falls	1950	5	45 –11.25	66,667	Q
Jefferson	1/57	5	75 –23	3,000	Q
Keokuk	1/52	7	37.5 –11.25	202,000	M, *Q*
Knoxville	6/49*	4	37.5 –18	26,667	B
Manchester	1948	5	33.75–11.25	6,667	Q
Marshalltown	6/54	7	60 –10	400,000	M, *Q*
Mason City	8/56	4	35 –10	84,000	Q
Maxwell	7/52	4	112.5 –18.75	6,667	Q
Muscatine	8/54	4	18 – 4.5	150,000	M
Newton	1952	12	55 – 9	1,500,000	M
Oelwein	7/54*	7	40 – 9	54,000	M

* Items marked with asterisk (*) were from communities indicating that a rate increase was then being considered.
† Range reflects rate from highest step (minimum use) to rate at the lowest step.
‡ Key: M—monthly; B—bimonthly; Q—quarterly. Those in italics indicate billing for majority of services (residential and often others).
§ During the 1930's.

Water Rates in Iowa Communities

7	8	9	10	11	12
			Quarterly Rate—$		
Penalty or Discount‖	Min. Bill *$/qtr.#*	Allowance on Min. *cu ft/qtr.*	3,000 cu ft	30,000 cu ft	300,000 cu ft
P-5	3.75	750	15.00	118.20	982.20
P-10	4.20	1,200	9.69	59.55	408.75
P-10	3.60	800	12.75	95.25	804.00
P-10	2.00	620	10.20	50.70	275.70
N	3.75	**	8.34	61.65	390.30
D-10			9.72	51.38	430.46
D-5	3.57	1,500	7.41	62.70	439.47
D-10	1.35	370	10.80	57.51	349.11
	2.30	570	10.73	49.70	286.20
N	5.43	1,115	14.63	87.75	560.17
P-10	2.40	600	9.30	78.60	561.60
N	2.10	700	9.00	52.20	376.20
D-10	1.80	500	8.10	46.35	410.85
N	2.00	667	9.00	72.00	372.00
N	3.25	900	10.80	69.30	528.30
D-10	2.70	300	13.95	111.15	1,083.15
P	6.00	925	19.50	112.50	667.50
P-10	3.00	1,110	7.25	53.98	519.73
P-5	3.75	1,245	8.54	44.08	250.08
D-10	1.46	535	7.31	34.79	171.81
P-$0.25	3.60	600	14.40	101.80	965.80
N	3.00	600	9.72	58.02	342.42
P-5	2.25	467	13.13	80.50	425.50
P-10	3.00	400	14.34	76.44	697.44
D-$0.50	2.20	600	10.75	62.65	380.20
N	3.00	800	11.25	90.00	576.00
P-10	3.00	893	9.75	44.00	347.75
P-10	3.00	500	13.20	73.20	418.20
P-10	3.00	867	10.50	54.00	351.00
P-25	3.00	267	15.75	73.25	579.50
D-10	3.00	1,667	5.40	27.00	159.30
N	3.30	600	11.16	70.86	412.86
P-10	3.00	750	8.94	45.58	293.40

‖ Key: P—penalty; D—discount; N—net. "P-5" means 5 per cent penalty.
\# Reflecting discounts where offered.
** No limit.

TABLE 1

1	2	3	4	5	6
Community	Date of Last Rate Adjustment*	Rate Steps		Quantity Above Which Lowest Rate Applies *cu ft/qtr.*	Billing Period‡
		No.	Range *cents/100 cu ft*†		
Orange City	1955	4	56.7 – 7.5	14,000	Q
Ottumwa	3/54	4	36 –11	1,080,000	M, *Q*
Rock Rapids	1/56	5	36.75–14.7	13,333	Q
Scranton	1950		75 –18.75	2,133	Q
Sioux Center	3/56	7	81.25–18.75	501,000	Q
Sioux City	11/56	6	35.4 –10.5	20,000	M, *Q*
Spencer	10/49*	7	40 – 4	280,000	Q
Storm Lake	1956	7	33.75– 8.25	200,000	Q
Washington	1956	5	37.5 –15	53,333	Q
Waterloo	1910	9	25 – 6	120,000	M
Waverly	1936*	6	30 – 7.5	6,000	Q
Webster City	4/55	7	40 –12	60,000	M
W. Des Moines	1953	5	48.9 –22.5	13,333	M, *Q*
Winterset	1948	6	50 –12	15,000	Q

* Items marked with asterisk (*) were from communities indicating that a rate increase was then being considered.
† Range reflects rate from highest step (minimum use) to rate at the lowest step.
‡ Key: M—monthly; B—bimonthly; Q—quarterly. Those in italics indicate billing for majority of services (residential and often others).

cubic feet per quarter for eleven steps from 1,000 to 300,000 cu ft per quarter. Rates for three steps are shown in Table 1 (Cols. 10–12). Prompt payment was assumed, thus taking advantage of discounts where offered and avoiding penalties for late payment. The charges for all steps are shown in graphic form (Fig. 1). Mean, or average, values were not calculated since they are so often distorted by a few extremes; rather, the summary bar graphs shown indicate the range of the data, the pattern of most common occurrence, and the median value, or midpoint—in this case, the charge which is less than that charged by half the cities and more than that charged by the other half. Median charges for 1,500 and 1,800 cu ft per quarter (typical range of residential use) were $5.50 and $6.40, respectively. Only one of the 47 cities reporting was on a flat-rate basis, with residential customers largely unmetered.

No trends in rates by size, treatment, or other such factors were apparent, because of the extreme variations in supply and treatment for those reporting.

The range from the highest to the lowest charge for any quantity was mostly 3:1 or 4:1. There are some questions that arise from this fact. Do total costs of supplying water to the consumer actually vary this widely? Surely not; a range of 2:1 would seem a strong variation. Are those cities at or near the median rate charging approximately the correct rates? The weight of numbers would so indicate.

Water Rates in Iowa Communities (contd.)

7	8	9	10	11	12
			Quarterly Rate—$		
Penalty or Discount‖	Min. Bill $/qtr.#	Allowance on Min. cu ft/qtr.	3,000 cu ft	30,000 cu ft	300,000 cu ft
P–10	4.25	750	12.13	42.63	245.13
P–5	4.50	1,200	10.80	79.20	463.20
D–2	2.50	667	9.31	54.39	451.29
P–10	3.00	400	9.33	59.95	566.20
P–10	3.25	400	11.60	82.70	602.75
P	4.25	1,200	8.30	47.70	331.20
P–10	3.00	667	12.25	64.00	348.00
N	2.25	667	8.88	63.00	386.00
P	3.00	800	10.50	77.50	491.25
P–10	2.25	900	6.55	43.05	223.02
P–10	1.00	333	6.97	28.80	231.30
P–10	3.60	900	10.80	70.50	405.90
N	3.00	613	14.64	93.76	701.26
P–10	2.50	500	8.25	45.25	369.25

‖ Key: P—penalty; D—discount; N—net. "P–5" means 5 per cent penalty.
\# Reflecting discounts where offered.

If so, are those with rates twice as high and more charging excessive rates and piling up unwarranted profits at the expense of their customers? This is particularly hard to swallow, since utilities by nature rarely are able to show a profit regardless of their accounting method. Assuming that those with "high" rates are showing a profit, where does this leave those with rate schedules bringing in ½ or less revenue per cubic foot delivered?

Rate Schedules

Concerning the rate schedules themselves, for the cities reporting, the number of steps from the first rate (for minimum use) to the last quoted step (for all over a specified amount) ranged from three to twelve. Schedules with four, five, and seven steps were most common, each reported by ten to twelve cities (Cols. 3 and 4, Table 1). The first step was most generally in the range of 30–50 cents per 100 cu ft, while the last step ranged most commonly from 8–16 cents (Fig. 2). Eight of the last steps were less than 8 cents, three of them less than 5 cents. The ratio between top and bottom step ranged from 1½:1 to 10:1, but was most often in the area of 2½:1–4:1.

With regard to the amount of change per rate step, a wide variation was noted, from a decrease of 54 cents per 100 cu ft in only three steps to a decrease of less than 10 cents per 100 cu ft in five steps. Decreases of 4 and 5 cents per rate step were most common.

The widest variation of all appeared in the quantity limits of the quoted rate schedule—the amount in excess of

which the last or lowest rate step applied. This range was 2,000–3,000,000 cu ft per quarter, with a median value in the range of 50,000–60,000 cu ft per quarter.

Here again some questions arise. Is there a proper or ideal number of rate steps? Would there also be a reasonable ratio for the upper and lower levels of a typical rate schedule? How far should the steps go, as far as quantity is concerned?

The author is by no means implying that twelve rate steps are too many, or that decreases of 2 cents or less per 100 cu ft are too slight, but is impressed by the wide variety shown in the makeup of the rate schedules reported and in the rates themselves. They apparently reflect much originality, with little concern with schedules of neighboring utilities. The question remains, however, of whether there are some basic guiding principles in establishing rate schedules which would make billing and bookkeeping simpler, make explanations to customers simpler, and insure that all water customers pay in reasonable proportion to the cost of providing service to them.* The old story of the major user paying less than cost is all too

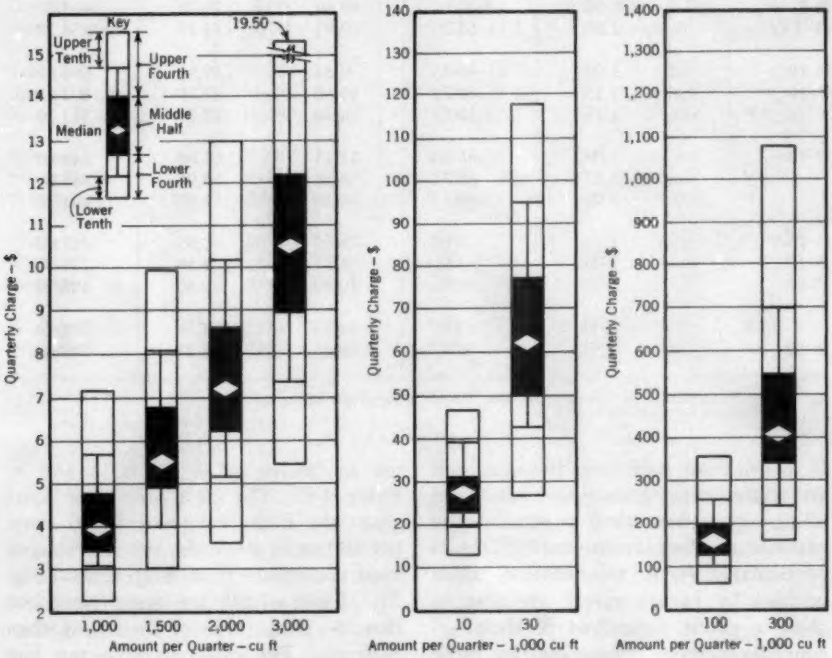

Fig. 1. Quarterly Rates of Iowa Utilities

Bar graphs show the range of charges by 47 Iowa utilities for the designated amounts of water consumed.

* There are: in *Water Rates Manual—M1.* Am. Wtr. Wks. Assn., New York (1954)—Ed.

familiar, especially with the smaller water utility.

Billing Features

The minimum charge on a quarterly basis ranged from $1 to $6 for residential service, ranging most commonly from $2.25 to $3.50. For 1-in. meters, it was in the range of $6–$9.

The allowance on the minimum bill, again for a small meter, ranged from 267–1,667 cu ft per quarter, but was most commonly 600–900 cu ft per quarter. Here again the question could be posed as to what proper or reasonable allowances and minimum charges should be.

Penalty provisions for late payment were much more common than discounts; 10 per cent was by far the most common figure for both.

Twenty-seven cities reported the use of cubic feet, while twenty, many of them smaller towns, were reading and billing on a gallon basis. It seems likely that a good deal of confusion and puzzlement on the part of utility customers could be saved by dropping the use of cubic feet in water sales.

Quarterly billing, especially of residential customers, was by far predominant, which leads to the question of why it is necessary to bill small users more often than that and whether the added cost of reading the meters, billing, and collecting on a monthly basis is justified.

Rate Adustments

Since 1954, this group of 47 cities had enjoyed an average of seven rate increases a year; several cities had increased rates more than once since 1950. Yet, six of the 47 had not adjusted rates in more than 20 years. The earliest one reported dated back to 1910.

An analysis of rate schedules for those cities going without a rate increase for over 20 years indicated that, as a group, their rates were comparatively low, minimum charges were lower, and the last quoted rate step was quite low. This is reasonable considering water utility cost indexes,

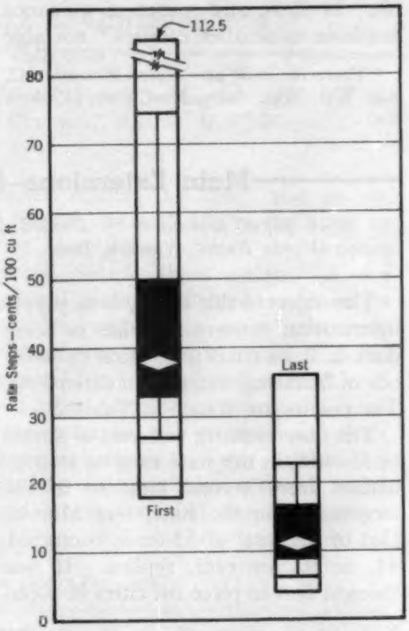

Fig. 2. Rate Steps of Iowa Utilities

The left-hand bar shows the range of charges in the first rate step; the right-hand bar, in the last rate step.

which doubled from 1913 to the mid-twenties, then have nearly tripled again since that time. How is it possible to exist on rates established 20–40 years ago?

Conclusions

A sampling of data on rates and billing procedures for Iowa water utili-

ties has been presented; probably there is nothing here that could not be found in any similar group.

One of the striking things is the large variations in rates, and rate features, leading to the question of whether there are certain features, minimum charges, or rate levels which should be incorporated in a rate schedule. Is there any source of guidance available to smaller utilities * not able to procure specialized professional advice on such matters?

As stated, Iowa has no utility regulation of any kind. Granting that any such control is a step toward regimentation, however mild, and loss of certain freedoms, it might be justified by the aid given to all utilities in remaining financially sound. Aid is particularly needed by utilities which, for a variety of local reasons, are falling far behind in not only revenue but also in such things as main extension policies and other management areas.

* There is: in *Water Rates Manual—M1.* Am. Wtr. Wks. Assn., New York (1954)—Ed.

Main Extensions—*Donald Y. Caldwell*

A paper presented by Donald Y. Caldwell, Mgr., Newton Water Works Board, Newton, Iowa.

The object of this survey is to supply information concerning what is being done in Iowa cities in relation to methods of financing water main extensions. The results are shown in Table 2.

The questionnaire was sent to a total of 55 cities in the state ranging in population from several hundred to the largest city in the state, Des Moines. Out of the total of 55 cities contacted, 44, or 80 per cent, replied. It was thought best to place the cities in population categories for the purpose of this report. The breaking points of the categories are purely arbitrary.

Table 2 lists three methods of financing water main extensions:

1. Financed entirely by the utility
2. Financed either entirely or in part by the benefited parties with some provision for a refund of all or part of the sum advanced (This refund is sometimes in the form of cash, and some-

TABLE 2

Methods of Financing Main Extensions in Iowa

Population Groups	No. of Cities Contacted	No. of Replies	Methods of Financing		
			Entirely by Utility	By Benefited Parties*	
				With Refund	Without Refund
Over 50,000	6	3	0	1	2
30,000–50,000	4	4	1	3	0
20,000–30,000	4	3	2	1	0
10,000–20,000	6	5	3	2	0
5,000–10,000	17	14	8	5	1
5,000 or less	18	14	9	3	2

* Entirely or in part.

times is based on water receipts from the property.)

3. Financed either entirely or in part by the benefited parties without any refund.

There was an attempt made in the questionnaire to differentiate between extensions in new subdivisions and extensions in other than new subdivisions, but in listing them in Table 2 no such differentiation is made.

The results of this survey show a tendency on the part of the growing medium-sized and larger cities to place at least some of the burden of financing on someone other than the utility.

Job Classifications—Mark E. Driftmier and George C. Ahrens

A paper presented by Mark E. Driftmier, Supt., Burlington Munic. Water Works, Burlington, Iowa, and George C. Ahrens, Mgr., Water Works, Ottumwa, Iowa.

In preparing the questionnaire on job classifications, hours, and wages, various classes of work were indicated by code numbers. This was done in order to set some standard method whereby personnel in both large and small utilities could be classified similarly. By subdividing each number, specific jobs within a department could be classified.

For example, meter department employees were listed generally as "5." Meter setting was classified as "5a," meter reading as "5b," and so on. Likewise, a small utility with one or two men doing overall maintenance would classify them under "12" for general plant maintenance. In larger utilities, where jobs were more specialized, the personnel could be listed under "12a," "12b," and so forth.

For the purposes of this report, however, material was grouped into three general categories. Table 3 shows, according to population groups, hours and wages for: [1] the manager or

TABLE 3

Hours and Wages of Iowa Utility Personnel

Item	Hours Worked per Month	Salary Range $/month	Item	Hours Worked per Month	Salary Range $/month
3,000 or less pop.			15,000–25,000 pop.		
Managers	190–234	300–550	Managers	173–208	358–600
Supervisors	*	*	Supervisors	173–190	370–425
All others	156–234	190–400	All others	173–208	251–405
3,000–8,000 pop.			Over 25,000 pop.		
Managers	173–234	285–475	Managers	173–190	615–750
Supervisors	190–208	230–365	Supervisors	173–190	350–584
All others	173–303	150–360	All others	173–190	210–486
8,000–15,000 pop.					
Managers	190–208	400–600			
Supervisors	*	*			
All others	173–208	200–333			

* Insufficient data.

executive; [2] the supervisory personnel; and [3] all other employees, regardless of responsibility—office work, treatment, distribution, or maintenance.

Some response was received to questions in the survey form concerning types of wage increases. To the question of whether personnel were given flat-rate increases, 29 answered "yes" and six answered "no." Ten replied "yes" to the question of whether they gave percentage increases, whereas sixteen answered "no."

It appears that flat or across-the-board increases of a specific amount per week or month is the common practice, even though the practice has resulted in unbalanced wages for various job responsibilities. This would have been partially avoided by percentage increases. Perhaps job evaluation should be given more consideration and study.

Reprints Available

Reprints of the following articles, published in the JOURNAL during the preceding year, will be available from the Association in small quantities, at the prices noted, until the present stock is exhausted. Order by reprint number and author's name from: Order Dept., American Water Works Assn., Inc., 2 Park Avenue, New York 16, N.Y. Prepayment required on orders under $2. (Note: This list does not include standards and similar documents kept permanently in stock.)

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Regulatory Problems of Privately Owned Water Utilities

John D. Reader

A paper presented on Apr. 26, 1957, at the California Section Regional Meeting, Santa Monica, Calif., by John D. Reader, Utilities Engr., Calif. Public Utilities Com., San Francisco, Calif.

MANY of the smaller water utilities under jurisdiction of the California Public Utilities Commission have few, if any, occasions to consult with the commission staff regarding the laws, general orders, or procedures applicable to their operations. It may be that some utility officials look upon regulation by a state agency with something less than complete enthusiasm and are more interested in their day-to-day physical operations than in keeping the records prescribed by the commission. These records, however, are necessary for evaluation of operations and for logical analysis of earnings. State law requires that rates charged shall be just and reasonable, and proper records are essential to such determination.

Some utility managements find it unnecessary to consult with the commission or its staff until there is a customer complaint or until some question arises which involves the application of tariff rates or rules. From experience, however, it has been found that commission engineers can be of considerable assistance to utilities many times. For example, the accounting resulting from the application of a water main extension rule can be complicated and, if the accounts do not correctly reflect the main extension agreements, the earnings picture may change materially when correctly set up on the books of the utility and properly reflected in its earnings base.

Some smaller utilities may delay starting the formal procedure involved in seeking adjustment of rates because they believe the preparation and presentation of necessary evidence to be a difficult task. When the rate adjustment is finally sought, a very high percentage increase is asked which is more disturbing to customers and perhaps more difficult to justify than promptly sought, more equitable rate adjustments. Rate changes necessary to keep a utility in healthy financial condition are usually in the public interest because, too frequently, service is allowed to deteriorate if the rate of return drops. If service falls below acceptable and reasonable standards this can lead only to difficulties. Furthermore, in a state where population is growing continually, utilities must maintain reasonable earnings to be able to finance the expansion of or replacements of plant which are required.

Informal Investigations

During the past year, the staff of the California Public Utilities Commission has initiated a policy of making informal field investigations of water utilities which have not been before it in formal proceedings for a number of

years. This permits a check of compliance with commission orders and requirements and information on whether utilities have properly applied their tariffs and rules. It also enables officials of the utilities to discuss with the commission representative any regulatory problems which they may have and to make use of the experience of the staff. Quite frequently the problem is one which has already been studied and solved by another utility—and this experience can be helpful. Where it is considered appropriate, a representative of the commission's utilities finance and accounts division accompanies the engineer from the hydraulic branch to check the records maintained by the utilities at their offices. Such informal staff field investigations have now been temporarily suspended in order to concentrate personnel on the heavy load of pending formal matters—particularly in the Southern California area. It is expected that the program will be resumed soon.

Contractual Arrangements

The sample form of contract or agreement between the utilities and the applicants for main extensions in effect and on file with the commission (as a part of the tariff schedules) sometimes gathers dust in the files of small utilities for so long that when an individual or a subdivider does apply for service, the existence of the sample form is often overlooked. General Order 96 of this commission provides that "no utility . . . shall hereafter make effective any contract or arrangement for the furnishing of any public utility service at rates or under conditions other than the rates and conditions contained in its tariff schedules on file and in effect at the time unless it first obtain the authorization of the commission to carry out the terms of such contract or arrangement." Another section of the order provides that any utility having sample forms of contracts or agreements on file may enter into such agreements without first obtaining the authorization of the commission—provided that any such contract does not deviate from the provisions of the sample form on file and it is not more restrictive or different from the filed main extension rule of the utility.

Any utility which does not have on file a sample form for such agreements must first obtain commission authorization for each agreement. Where the individual or a subdivider agrees with a utility to provisions which result in deviations from the utility's filed main extension rule, an appropriate application must be submitted which seeks the authorization of the commission for deviation from the provisions of the rule. Such an application must show complete data for justification of such deivation. The rules of procedure of this commission, a general order, and the main extension rule provide for such applications. In general, main extensions into new subdivisions require that the utility provide the water supply and standby plant facilities and that the subdivider advance the cost of the mains, services, and fire hydrants if requested or required by public authorities. Estimates of cost advanced are required to be adjusted to the cost actually incurred.

The water main extension rule was adopted by the commission in a decision—dated Sep. 28, 1954—which required that the rule be filed by all water utilities. It applies to both individual and subdivision extensions. With reference to extensions to serve individuals, the rule provides that the

utility shall not charge the individual for the cost of a main *more than* 4 in. in diameter—except where required by the special needs of the applicant. Recently, a utility installed a main larger than 4 in. at a place where, because of the geography of the area, it could not expect other individual customers to locate, and charged the individual for the estimated cost of a 4-in. main, as permitted in the main extension rule. The rule further provides that where the utility has provided some excess capacity, the individual is not entitled to refunds which might accrue from either continuous or lateral extensions. Although the rule provides that a utility may charge an individual for not more than a 4-in. main, circumstances may indicate that a 2-in. main would have been adequate to serve the individual and all intervening potential customers. To require an individual to pay for a 4-in. main when there is a possibility of obtaining a refund may be one thing, but where there is little or no possibility of a refund, a utility must be careful to charge the individual only the cost of a main which is adequate to serve the individual and any potential intervening customers.

Fire Protection Service

Other main extension problems have mainly involved the very small subdivider, where relatively large mains were required by the utility in order to provide adequate fire protection. Through discussion with the fire protection agency, such problems can usually be brought to a settlement which may result by a simple relocation of proposed fire hydrants. The staff of the commission encourages the subdivider and the utility to provide fire protection, but there are often alternatives which can save many dollars and, at the same time, provide fire protection service to the satisfaction of the local fire protection agency.

Backup Facilities

Some misunderstanding of the purpose and application of certain portions of the main extension rule has come to the commission's attention. The rule provides that: "If additional facilities are required specifically to provide pressure or storage exclusively for the service requested, the cost of such facilities may be included in the advance upon approval by the commission." The misunderstanding has arisen with utilities which have not developed water supplies adequate for the growth in customer requirements and which, as a result, have been ordered to cease serving new subdivisions. On occasion they have assumed erroneously that it is the subdivider's responsibility to develop needed additional water supplies. This results in the subdivider's paying for the greater part of the cost, because the repayment of 22 per cent of revenue for 20 years was designed only to return the cost of mains, services, and fire hydrants over a period of 20 years—not the cost of additional facilities.

Rule Deviations

If the commission has issued an order to cease serving new subdividers, the utility must petition the commission for modification of such restraining order when it believes it has developed the necessary additional water to serve additional customers safely. In several instances, such petitions have included a request to deviate from the main extension rule for purposes of allowing the inclusion of storage or supply facilities in the subdivider's advance. It is easy to understand how

the utility feels—that is, that the subdivider should advance the cost of some of these facilities because they are one of the primary sources of the continual pressure for increased investments in water facilities for a growing system. Within reasonable limits, however, the utility has a responsibility to provide the necessary supply and storage facilities in the area it undertakes to serve. The main extension rule was intended to provide for the complete repayment of subdividers' main extension advances if the subdivision development reaches a density of water customers not too different from that of the balance of the water system for which the rates were established. The utility may limit its area of service to its certificated area—or its presently dedicated area if larger—or, if the utility has a constitutional franchise because it or its predecessors were operating in this area before May 1, 1917, it may designate the area to which it will continue to render service so long as all areas then served are within the area to which it wishes to limit service. A utility may extend service beyond such limits but, if it does so, service must be rendered under the same rates and rules that are in effect for its existing customers.

During 1956, the commission received an application for authority to deviate from the main extension rule for a subdivision which was built upon filled ground. The utility felt that the maintenance of water mains in filled ground might be excessively costly both from the point of view of settling and from possible earthquake damage. The commission issued its decision *ex parte* in this matter, authorizing the utility to deviate from the main extension rule as requested. Petition for rehearing was filed by another subdivider group planning a subdivision on filled land in the same general area. It was alleged that "a great deal of filled land in the marsh area in question as well as the rights of a substantial number of persons are involved." A complete investigation and public hearing was asked to determine if any deviations from the filed main extension rule should be authorized under these circumstances. After review early in 1957, the commission issued its order denying petitioner's request for rehearing on the grounds that petitioners were not parties entitled to apply for rehearing. The commission did, however, reopen the matter so that "petitioners should have an opportunity to be heard with respect to the subject matter."

During 1957, the commission received another application from the same utility for an order of the commission in settlement of a main extension dispute involving service in the subdivision being developed by the above-mentioned petitioner. The issues in this application are not all the same as those requested in the applicant's first application for a subdivision located on filled land. These matters have not yet been heard.

New General Order

"Rules Governing Water Service Including Minimum Standards for Design and Construction," a general order of the commission, was issued by decision on Jun. 12, 1956. The actual experience of many of the public and private water utility system owners and operators was of great assistance to those participating in the development of this general order and, to a great extent, accounts for its apparent success.

There are several provisions which would be of interest to every public water utility. At the commission the order is referred to mainly in connection with applications for certificates of public convenience and necessity in the construction of new systems or the extension of existing ones. It then becomes the basis for evaluating the proposed installation or extension. The administration of this general order has so far caused no particular difficulties.

Section 1 provides that its "purpose is to promote good public utility practices; to encourage efficiency and economy; and to establish minimum standards to be hereafter observed in the design, construction, and operation of water works facilities by water utilities operating under the jurisdiction of the commission." It goes on to state that it is not the intent of the general order to require the replacement or abandonment of any utility water system "prior to the expiration of economic utilization of facilities in use" unless so directed for any facility found to be inadequate. This section also provides that each utility shall maintain system maps and records containing specified details. Certain portions of the general order apply to ditch systems.

Both Sections 1 and 2 set forth the basis for many of the rules filed with the commission by water utilities, which rules govern their relations with their customers. Section 2—standards of service—also establishes the limits for operating and static pressures and requires that all public utility water systems have some means of measuring each source of water supply.

Standards of design are covered in Section 3 which establishes certain minimum pipe sizes, water supply requirements, and some general material specifications which include steel pipe coatings and linings and minimum gage requirements for wall thicknesses of steel pipe. Section 4—standards of construction—establishes that the minimum depth, with certain exceptions, is 30 in. for mains and 18 in. for services. The installation of valves at regular intervals and the size of service pipes are also covered.

The main extension rule ordered to be filed by each water utility under the commission's jurisdiction is referred to in Section 5 in which is defined the responsibility of the utility and the customer with reference to service connections.

Section 6—measurement of service—covers, in general, the subjects set forth in the rules filed with the commission by most privately owned utilities. One additional provision pertaining to the periodic testing of water meters provides that meters shall not remain in service longer than 10 years unless the utility makes a showing by letter that some other period is economically justified, and that any utility electing to determine some different period or periods for its system or systems must so advise the commission before Jun. 30, 1957. It also provides that upon receipt of notice of such intention a utility has a 5-year period beyond the effective date of the order in which to show the necessity for selecting the different period.

The commission's general order entitled "Filing and Posting of Tariff Schedules"—is referred to in Section 7 and sets forth certain information which must appear on the billing forms. This section, as well as part of Section 6, sets forth the bases for other rules filed with the commission by water utilities under its jurisdiction.

Water Shortage Problems

The commission's staff has been concerned about the continued, rapid, and extensive expansion of several of the existing water utilities as well as by the question of granting certificates of public convenience and necessity to new utilities in areas where the water supplies are known to be extremely limited, deficient, or uneconomical to develop. A recently adopted general order goes into standards for service at considerable length in order to provide new utilities with some minimum basis with which to gage and evaluate the water supply to be developed for their particular situation. Recent decisions have, on several occasions, required water utilities to obtain first additional water supplies before undertaking to serve any new or additional subdivisions. A showing of adequacy has been required to be made by formal filing—many such filings having been supplementary to the case or application which initially established the restriction. Proceedings of this nature have generally permitted the extension of service to individual applicants. The commission, however, now has an application from one of the larger utilities which asks that it not be required to serve any additional customers until such time as additional water supplies are brought into the area in such quantities that the maximum demands of all existing and potential customers within the present area of service have first been met.

Some problems involving an apparent lack of water supply, upon analysis, turn out to result from no more than inadequate transmission or distribution system capacity or inadequate pumping facilities. Minimum distribution system requirements are also set forth in the order for the purpose of providing design criteria which should result in a satisfactory water system. This procedure of placing the utilities on notice by general order of the commission in advance of applying for authority to serve an area minimizes the possibility of rebuilding whole systems or portions of systems on the assumption by the utility that the service planned for would be satisfactory because it is in the public interest.

A few public utilities within California seem to have more or less perpetual water shortage problems. The Public Utilities Code provides that the commission, after a hearing and a finding that the addition of more customers will injuriously reduce the supply of water to existing customers, may order that no additional customer be served until such an order is modified or vacated. Such water shortage problems seem to be characteristic of two types of systems—those located in areas which have limited underground or surface water supplies, where it does not appear to be economically feasible to bring water into the community and those in areas where underground or surface water supplies are available, but where rapid growth or poor management has caused the existing facilities to become inadequate.

In either type, the result is the same and the commission must usually issue its order limiting the expansion of service until improvements can be installed and placed in operation, until more water is developed locally, or until it is imported into the area. Most problems seem to fall in the second category—where water is available but has not been adequately developed. Usually, an order limiting service in such situations will require the improvements necessary and provide for

modification of the order upon a sufficient showing that additional water supplies have been developed and now meet all requirements. The wide fluctuation in rainfall sometimes causes this problem and may be the factor which management has the most difficulty in properly considering.

Conclusion

It is recognized that the financing of small water utilities is a major problem and also that some confusion is caused by the existence of both publicly owned and privately owned water systems which operate side by side. Although the main extension rule is intended to relieve the privately owned water utility from the risks inherent in the development of subdivisions, the large-scale developments throughout California have resulted in the use of the main extension rule by the utilities as a means for financing over half the cost of the additions to many of the smaller water systems.

Revision of Standard for Steel Pipe Fitting Dimensions (AWWA C208)

On Dec. 31, 1957, the AWWA Board of Directors approved the following revisions to AWWA C208–55T—Standard for Dimensions for Steel Water Pipe Fittings:

The following paragraph is to be added to the explanatory material preceding the tables and drawings:

> *Weight.* The weight of plain-end fittings shown in Tables 1 and 2 can be determined by multiplying the length of the fitting as given in the tables by the weight per foot of straight pipe of the same size and thickness.

In Table 2, under "Elbows," the columns headed "*R2*," "*R3*," and "*R4*" are to be omitted, and the words "*or R5*" are to be deleted from the "*Q*" column. In the footnote to Table 2, the sentence "Elbow radiuses are approximate" is to be omitted, and the following sentence is to be inserted at the end of the note:

> Six inches has been added to all ends of fittings for mechanical couplings.

In addition, formulas are to be inserted in Fig. 1 and 2 to show the relationships between the elbow dimensions and their radiuses, and the drawings are to be clarified.

The revised standard remains Tentative and will be designated "C208–57T."

Use of Gunite in Reservoir Construction

Russell C. Kenmir

A paper presented on Nov. 1, 1957, at the California Section Meeting, San Jose, Calif., by Russell C. Kenmir, Chief. Engr., James M. Montgomery, Cons. Engr., Pasadena, Calif.

FOR years there have been popular misconceptions as to where and how gunite should be used in the construction of reservoirs. In this article, an attempt is made to show under what conditions gunite can be used to advantage, how it should be applied, and where it should not be used. The Gunite Contractors Association (*1*) defines gunite as "a proportional combination of sand and Portland cement which is mixed and pneumatically conveyed in a dry state to a nozzle, where hydration takes place immediately prior to expulsion. The terms 'pneumatically applied mortar' and 'pneumatically placed concrete' are often used to describe gunite." *

For some reason, many people, other than engineers, have come to think of gunite as having such desirable properties as absolute watertightness and a strength many times that of poured concrete. Actually, tests show that properly applied gunite is relatively watertight, if cracks do not develop, and has a compressive strength at the age of 28 days of 3,000–4,000 psi which is comparable to a good quality of poured concrete. Since gunite reservoir linings are generally only 2½–3½ in. thick, the gunite cannot possibly have very much strength when subjected to beam action, such as takes place at soft spots in the subgrade or where the gunite is placed on subgrades composed of expansive soils.

One of the most common characteristics of gunite linings is the tendency for the material to crack badly upon drying out. In reservoirs where the water surface is kept considerably below the high-water level for long periods, the lining is especially susceptible to cracking. Such cracking can be minimized by providing an adequate amount of reinforcement steel, by using weakened plane joints to permit contractions, by using great care in the curing of the gunite, and by filling the reservoir promptly after the curing period.

Concepts of Reservoir Design

In order to determine where gunite linings may be used to advantage and at the same time recognize the limitations of this type of lining, it is necessary to understand certain basic concepts in reservoir design.

Although vertical-wall reservoirs are generally regarded as being more durable than sloping-side reservoirs, they are considerably more expensive. Therefore, due primarily to economic reasons, the sloping-side reservoir is much more common in mild climates

* Although "Gunite" is a trade name of the Cement Gun Co., Allentown, Pa., the word is often used as a common noun in the construction industry.

than vertical-wall reservoirs. In cold climates, where protection against frost must be considered, this is not necessarily true.

Sloping-side reservoirs may be of the so-called cut-and-fill type, constructed in such a way that the quantity of earth that is excavated approximates the quantity of earth required for the compacted embankment; or sloping-side reservoirs may be constructed entirely in cut with the excavated material wasted.

In any sloping-side reservoir, the lining should not be considered as a structural member. It serves as a protection against erosion and as a means of facilitating cleaning the reservoir. In reservoirs with impervious linings, the lining, of course, serves also as a means of insuring watertightness. If the lining were made heavy enough to perform as a structural member, it would increase the cost to such an extent that the usual economy in a sloping side reservoir would be lost.

Since no lining should be considered as absolutely watertight, the earth subgrade upon which the lining is placed should be firm enough when saturated to support without excessive deflection the load imposed by the water when the reservoir is full. Since most soils in their natural state have low bearing values when saturated, it is generally necessary, in order to provide proper support for the lining, to overexcavate the bottom and the cut portions of the sides and construct a compacted blanket. Reservoir embankments are in effect small dams and should be stable without reliance upon the lining.

Two general types of lining are used in sloping-side reservoirs, pervious and impervious. With the pervious type, a relatively impervious earth blanket over the entire bottom and slopes is relied upon for watertightness. Since underdrains are not generally used with this type of lining, the lining is made porous to relieve the hydrostatic pressure back of the lining at times when the water level in the reservoir is drawn down quickly. Where gunite is used as the lining, the required porosity may be obtained by perforating the lining. The perforations usually consist of 2-in. holes spaced about 5–10 ft apart both ways.

With the impervious type, the lining alone is relied upon for watertightness. If the subgrade is relatively impervious, underdrains usually are provided in order to relieve the hydrostatic pressure that may build up back of the lining as a result of a small amount of leakage over a long period of time. Generally speaking, underdrains are not provided if the subgrade is sufficiently permeable to absorb the probable maximum leakage. Impervious gunite linings should not be placed on expansive soils, such as adobe, for the reason that such materials swell when they become wet. The degree of swelling is a function of the degree of compaction—the greater the compaction, the greater the swelling when saturated. Since normal construction methods cannot insure absolute uniformity in the degree of compaction, the swelling which follows the saturation of soils of this type is far from uniform. This irregular swelling of the subgrade produces very severe stresses in the thin gunite lining and usually results in serious and extensive cracking.

Conditions of Use

Throughout the western states, there are a great many gunite-lined reservoirs in use today. In areas where

the embankments are found to be stable on steep slopes and at the same time are relatively impervious, gunite has a great economic advantage over other competitive materials. Some of the large covered reservoirs in the water distribution system of San Diego, Calif., are examples of this type of construction, with embankments for these reservoirs on slopes as steep as 1½:1. Two large uncovered distribution reservoirs built by the Metropolitan Water District of Southern California are examples of those with embankments on 3:1 slopes. Figures 1–3 illustrate construction and one of the completed reservoirs.

In large reservoirs located in areas where the climate is mild and where it has been necessary to use 2:1 or flatter slopes in order to insure stability of the embankments, porous asphaltic concrete has been used extensively in place of gunite because of the lower cost of asphaltic concrete. In cold climates, it is doubtful whether porous asphaltic concrete would be very durable because of the freezing and thawing action. In small reservoirs it is difficult to maneuver spreading and rolling equipment. For this reason, porous asphaltic concrete offers little competition to gunite in the smaller reservoirs, such as those having capacities of 1–3 mil gal. Gunite has practically no competition with other materials in swimming pool construction.

Fig. 1. Orange County Reservoir, Brea, Calif.

Workmen are shown applying the gunite lining.

Poured concrete has become more competitive with gunite in connection with sloping-side reservoirs, as a re-

sult of recently developed equipment and materials. With the development of improved vibrating screeds, poured concrete can be placed on slopes as steep as 2:1 with reasonable assurance that the concrete will be dense and free from rock pockets. Watertightness at joints between concrete panels is more easily achieved now because of the excellent sealing materials which are available, including a variety of rubber and plastic waterstops for embedding in the concrete, and a synthetic rubber sealant (polysulfide polymer) for filling joint grooves.

Another competitive lining material which has become available in recent years is the so-called "asphaltic panel" or "asphalt plank." This material consists of two layers of felt, with a special asphaltic filler sandwiched between them. It can be obtained in thicknesses of ¼ in. and ½ in. The material can be laid directly on a carefully prepared and smooth subgrade. As far as cost is concerned, this material is competitive with 2½–3-in. gunite or 4-in. poured concrete. This material competes with gunite in sealing leaky reservoirs if the surfaces on which it is to be laid are fairly uniform and free from spalling. If the surfaces require a great deal of repair, then gunite should be less expensive for this purpose.

Fig. 2. Construction of Palos Verdes Reservoir, San Pedro, Calif.

This view shows applications of gunite in various sections of the sloping side.

Construction Methods

In the design and construction of gunite-lined reservoirs, several factors should be taken into account. First,

test borings should be made and a foundation report should be prepared by a competent foundation engineer who is experienced in reservoir construction. The test borings should extend 10–20 ft below the bottom of the reservoir. The report should include logs of the borings, physical characteristics of undisturbed samples of the soil, determination of shear strength, consolidation, compaction, percolation rate, moisture content, and density. The shear and consolidation tests should be made at the natural moisture content and at saturated moisture content. The foundation report should indicate the steepest slope on which the embankments would be stable. It is generally a good idea to give consideration in the report to the possibility of using either a pervious or an impervious lining in order that comparative estimates of the two types of construction may be prepared.

If the natural soils found on the site are relatively impervious when compacted, then a pervious lining could be expected to give the most economical design. If the natural soils found on the site are not relatively impervious when compacted, then it is advisable to make a cost comparison between pervious and impervious linings. The cost estimate for the pervious lining should include the cost of the embankment, including the compacted impervious earth blanket, and the lining itself. The cost estimate for the impervious lining should include the cost of the embankment, the lining itself, and any underdrains which may be necessary.

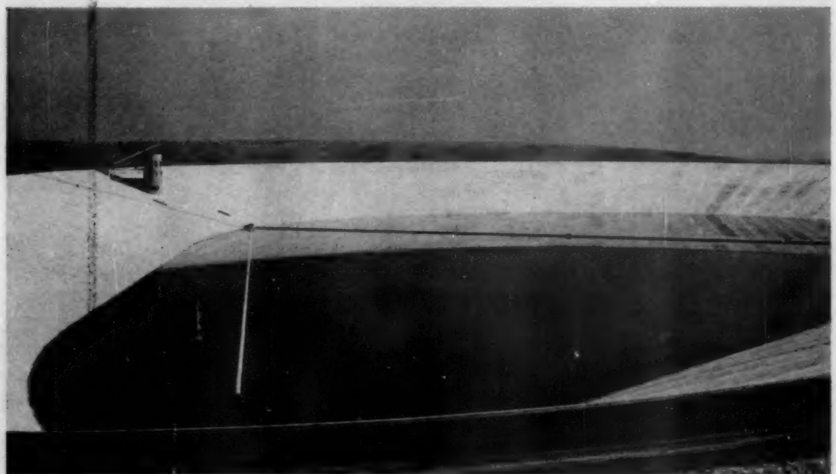

Fig. 3. Completed Palos Verdes Reservoir

This view shows the 3:1 slope on the embankments, which were lined with gunite.

Regardless of which type of lining is used, the compaction of all embankments should be given close attention. Compaction tests should be made by qualified technicians as the work progresses, and records of all tests should be filed. Any piping passing through or under the embankments should be

pressure tested and then encased in concrete. The encasement should cover the full width of the trench, and adequate cutoff walls should be provided. In order to provide a firm foundation for the lining, it is desirable to overexcavate the bottom and sides and backfill with compacted earth. Where the sides are overexcavated, the degree of overexcavation should be sufficient to permit small equipment to place and compact the backfill in horizontal layers. The backfill should be overbuilt at least 1 ft, measured at right angles with the slope, and then bladed off to the required grade.

If a pervious lining is selected, $2\frac{1}{2}$-in. gunite with only a nominal amount of reinforcing—about 0.25 per cent of the cross-sectional area in each direction—would generally be satisfactory. Since shrinkage cracks are of no particular consequence in a pervious lining, no weakened-plane joints with rubber sealant are necessary. Two-inch diameter perforations on 5–10-ft centers both ways are usually provided in a gunite lining intended to be pervious, in order to relieve the hydrostatic pressure back of the lining which might exist upon rapid lowering of the water surface. At each perforation, it is desirable to overexcavate the subgrade and place beneath the lining a block of porous concrete about 1 ft square and 4 in. thick to serve as a filter and thus prevent gradual erosion of the subgrade.

If an impervious gunite lining is selected, it should be very carefully designed. Unless the subgrade is quite pervious, it is advisable to provide an underdrain beneath the floor near the toe of the slope to prevent the possibility of hydrostatic pressure build-up in back of the lining. Such an underdrain should be designed to operate effectively as a filter. Unless this is done, there is always the possibility that a bad leak may result in washing the earth subgrade out through the underdrain, and thus undermining the reservoir. The thickness of the impervious gunite lining should be 3–4 in., and the steel reinforcement should be about 0.5 per cent of the cross-sectional area in each direction.

Reinforcement consisting of a combination of small bars, widely spaced, and wire mesh has some advantage over wire mesh alone, because it is more easily held in proper position. Where bars are used, however, the nozzleman must be careful to avoid occlusions of rebound beneath and along the sides of the bars. Weakened-plane joints, spaced about 20 ft apart in both directions, should be provided, and the joints should be sealed with rubber sealant. Although many people experienced in the use of gunite prefer to carry the reinforcement across all joints, the Portland Cement Association recommends against this. The author is of the opinion that if joints are provided, the reinforcement across the joints should be discontinuous.

Repair

Gunite has proved to be a very effective method of rehabilitating old reservoirs. For example, the Fontana Union Water Co. in California not only relined its old reservoir with gunite, but also increased the capacity by constructing a vertical cantilever wall of the material (see cover picture).

In the repair of old concrete- or gunite-lined reservoirs, there are very often loose plastered coatings and chunks of concrete that must be removed. Also, after all loose material has been removed, there usually remain numerous depressions which should be cleaned and filled with gunite or mortar. There is a difference of

opinion among engineers and contractors as to whether a new gunite lining in an old reservoir should be bonded to the old concrete. The Portland Cement Association favors breaking the bond. This can be done easily by spraying asphalt emulsion on the surfaces after all depressions have been carefully filled. If the bond is not broken, cracks are likely to appear in the new lining in the same locations as the cracks in the old lining. The remainder of the work should be done in the same manner as for new linings.

Reference

1. Gunite Specifications and Recommended Practice, Brochure G-55-57, Gunite Contractors Assn., Los Angeles, Calif.

APPENDIX

Typical Gunite Specifications

Typical specifications for gunite work as recommended by the Gunite Contractors Assn. are summarized as follows:

1. Proportions of cement and sand based on dry and loose volume: 1:4½.

2. Cement: Type I or Type II, ASTM C-150.

3. Sand:

a. Deleterious substances: not more than 5 per cent by weight.

b. Fineness modulus: 2.7 to 3.3.

c. Grading:

Size Sieve	Percentage by weight
Passing ⅜ in.	100
Passing No. 4	97–100
Passing No. 8	79–85
Passing No. 16	60–73
Passing No. 30	36–47
Passing No. 50	10–20
Passing No. 100	0–4

d. Moisture content: 3–6 per cent (sand and cement proportions should be corrected to provide for bulkage due to sand moisture content).

4. Minimum water pressure at nozzle: 15 psi above air pressure at nozzle.

5. Steel reinforcement:

a. Minimum area: 0.25 per cent of cross-sectional area in each direction.

b. Bar spacing: 2½ in. clear between bars.

c. Minimum lap of wire mesh: One mesh.

6. Minimum air pressure at nozzle:

a. Hose length up to 100 ft: 45 psi.

b. Hose length in excess of 100 ft: 5 psi for each additional 50 ft.

7. Rebound: May be reused as sand in quantity not to exceed 20 per cent of total sand requirement.

8. Construction joints: To be sloped to a thin edge and the edge thoroughly wetted before adjacent section of gunite placed. No square joints permitted.

9. Curing: To be damp cured for at least 5 days * after placing or may be cured by approved sealing compound.

10. Tests: Compression tests to be made on specially constructed 6-in. diameter by 12-in. high cylinders formed with ¾-in. square mesh hardware cloth. Cylinders to be shot with same air pressure, nozzle tip, and hydration as gunite in structure. Hardware cloth forms to be removed 24 hr after cylinders made, stored and tested in accordance with ASTM designation C-39. Required minimum strength at 7 days: 2,400 psi. Required minimum strength at 28 days: 3,000 psi.

11. Placing: Except when enclosing reinforcing steel, nozzle to be held at right angles to surface and at distances of 2½–3½ ft from surface being gunited.

* The author favors a 7-day water cure with no alternate for a curing by a sealing compound. For reservoir linings that are intended to be impervious, it is a good plan to sprinkle the gunite daily until the reservoir is filled with water, in order to reduce the possibility of shrinkage cracks developing.

In reservoir construction, a wood float finish of the gunite usually is considered satisfactory.

Ohio River Water Quality and Flow

Edward J. Cleary and David A. Robertson Jr.

A paper presented on Sep. 18, 1957, at the Ohio Section Meeting, Cincinnati, Ohio, by Edward J. Cleary, Executive Director, and David A. Robertson Jr., San. Engr., Ohio River Valley Water Sanitation Com., Cincinnati, Ohio.

FOR those charged with the task of unraveling a mystery—and the behavior of rivers often is mysterious—the attitudes of police detectives provide inspiration. Detectives rely upon relentless insistence and determination to get the facts. Likewise, those engaged with the mysteries of water pollution control also need the facts. Although the techniques for getting them may differ somewhat from those employed by detectives, success requires the application of similar qualities of determination, ingenuity, and painstaking attention to detail.

The purpose of this article is to describe how the Ohio River Valley Water Sanitation Commission (ORSANCO) has gone about the task of getting some basic data on quality conditions and flow variations. Because of current interest in establishing a national network of river quality stations, it was suggested that it might be useful to outline the commission's experiences with a monitor program that is now in its sixth year of development. In so doing, the following aspects of the program will be described: [1] genesis of the ORSANCO project; [2] methods of operation; [3] special monitoring activities; [4] assembly and evaluation of data; and [5] cost information.

The Ohio River Valley Water Sanitation Commission is an interstate agency, established in 1948 by a compact among the states of Illinois, Indiana, Ohio, Pennsylvania, New York, Virginia, West Virginia, and Kentucky. These eight states pledged a pooling of their resources and their police powers in a regional crusade for clean waters. To carry out this purpose a commission was created, consisting of three representatives from each state appointed by the governor of the state, and three representatives from the federal government appointed by the President of the United States.

The role of the commissioners is to promulgate interstate waste control regulations and to assert such powers as may be necessary for the enforcement of obligations outlined by the compact. For administration of commission functions, the eight states maintain a staff and headquarters at Cincinnati, the cost of which is assessed on a proportionate basis among them. The annual budget of the Commission for the past 2 years was $130,000; for the preceding 5 years, it was $100,000 annually.

Monitoring Project

The water quality monitoring project had its genesis in the fall of 1951. At that time the executive director requested commission authorization to initiate procedures for securing river

quality and flow information on a continuous and systematic basis. It was pointed out that in dealing with a dynamic river system like the Ohio—which includes 1,000 mi of main stem and 19 major tributaries—occasional stream surveys would hardly suffice for a proper diagnosis of quality conditions and the formulation of control measures. Furthermore, since there are thousands of industries and municipalities discharging wastes into the river system at various points, the problem of checking compliance with control requirements called for consideration of a systematic surveillance of stream conditions. And finally, through routine observation of quality variations at selected points on the river, it would be possible to institute a system for alerting downstream water users of abnormal conditions, as well as fixing responsibility for their origin.

From these considerations emerged the idea of establishing a network of monitor stations whose activities would be coordinated to produce a diary of facts about the river. How this ideal conception might be realized within the limitations of a small budget was something else. Thus, armed with more enthusiasm than money, the staff invited the interest of eleven managers of public and private water utility plants situated at strategic spots along the Ohio River. These men displayed a generous concern with river water quality—so generous, in fact, that they responded to an appeal to serve as volunteer, unpaid monitors.

This resulted in the organization, on Jan. 7, 1952, of the Water Users Committee of the commission and the formal beginning of the ORSANCO river quality monitor project. Each member of the committee agreed to furnish a twice-weekly analysis of Ohio River water; all this, and much more, has been faithfully done for the past 6 years and still continues. Incidentally, operations of the committee, which meets every 3 months with the staff for exchange of experiences on quality conditions, has been kept within the $1,000 allotted annually in the commission budget for this committee. The commission acknowledges with the greatest pride the contribution that has been made by the Water Users Committee, and looks upon its efforts as one of the most fruitful undertakings in the Ohio Valley program.

Two years after the volunteer monitor project had been initiated and its usefulness demonstrated, the commission budget permitted execution of the next step in the program. This was the establishment of six additional monitor stations near state lines and other strategic points where water plants are not located. The extension was made possible through a cooperative-contract arrangement with the US Geological Survey, whose long-time activity in basic-data assembly was a source of admiration and inspiration to the commission. The $5,000 made available by ORSANCO was matched with an equal sum by the USGS, and the six additional monitor stations were placed in operation in 1954.

In October 1955, the USGS contract was amplified to include analyses during low-flow months for certain trace constituents, notably in the heavy metals groups, at three places in the Ohio River. All of the USGS stations are located to take advantage of a happy arrangement with the Ohio River Division of the US Corps of Engineers, through which ORSANCO samples are collected by lock-and-dam personnel.

Beginning in October 1956, the cooperative agreement with the USGS

increased the number of stations from six to fifteen, bringing into operation nine additional stations on tributary streams. The USGS also maintains cooperative studies with the states of Kentucky, Ohio, and Pennsylvania. By coordination of these operations, data is secured from a total of 28 stations on the Ohio River and on tributary streams, thus providing multiple-purpose use of the results.

To sum up, data are now being secured from a network of stations at 43 locations. Fifteen of these stations are serviced by an enlarged Water Users Committee, and the remainder under the cooperative agreements with the USGS.

Methods of Operation

Keeping the diary of a river calls for a coordination of activities. This was initially undertaken by Robert K. Horton, assistant director of ORSANCO, and for the past year has been carried forward under the supervision of one of the authors, David A. Robertson Jr. Advice on many phases of the work is supplied by Harold W. Streeter, who serves as staff consultant.

Activities of the volunteer monitor stations are coordinated through the Water Users Committee. The members of the committee, each of whom operates one or more stations, meet every 3 months. This provides opportunity for discussion of analytical techniques, review of observations, and exchange of experiences. In addition, it offers a unique opportunity for ORSANCO staff members to become intimately acquainted with treatment problems at municipal and industrial plants as revealed by men who are concerned with the daily task of processing river water. This is an important dividend, since it provides the staff with practical background for the interpretation of the analytical data supplied by the committee members.

When abnormal situations occur on the river—as evidenced by a spill, detection of a slug of waste, or a fish kill—members of the committee alert ORSANCO headquarters by telephone. In turn, the headquarters keeps the monitor stations informed of unusual conditions. In this fashion a patrol system of great usefulness has been established. It serves two important purposes, providing a means for alerting downstream water plants of potential difficulties so that steps can be taken to cope with the situation, and facilitating the task of finding out where the trouble originated so that steps can be taken to prevent a recurrence.

There have been several occasions where the Water Users Committee monitor stations made it possible to bracket quickly a stretch of river and pin down responsibility for troublesome situations. One of these involved a thoughtless plant manager who ordered the bypassing of a recovery unit in order to make some repairs, with the resultant discharge of several thousand pounds of phenolic wastes and the temporary ruination of a water supply; prompt notification to downstream users about this slug of waste helped to minimize trouble at other places. Another incident concerned a shift foreman who shut down his operations one night but forgot to stop a pump that was filling storage tanks with oil. The tank began overflowing into a creek leading to the Ohio River. The following day a boat club notified the commission office that the river was a mess. The source of discharge was thus bracketed between two monitor stations and the information relayed to the appropriate state agency, whose field inspector found the location and

had the pumping stopped. It has been noted from experience in the Ohio Valley that virtually all spills, abnormal discharges, and other troubles occur between 5:00 PM on Friday evenings and noon on Saturdays.

The above is an accounting of the manner in which the volunteer monitor stations function. Concerning the USGS, at nine of their stations an arrangement has been made with the US Corps of Engineers for collection of samples. The Ohio River and some of its tributaries are fitted with navigation locks and dams, which are permanently manned. The sympathetic interest of the Corps of Engineers made it possible to secure the aid of personnel at the navigation locks and train them in the art of taking samples. Samples from other stations are secured through a contract arrangement with a local resident.

Samples are collected by a truck operating from the Columbus, Ohio, laboratory of the USGS, which makes the rounds on a semimonthly schedule. At a few remote stations where the extension of this routine pick-up service would be costly, the samples are shipped by rail.

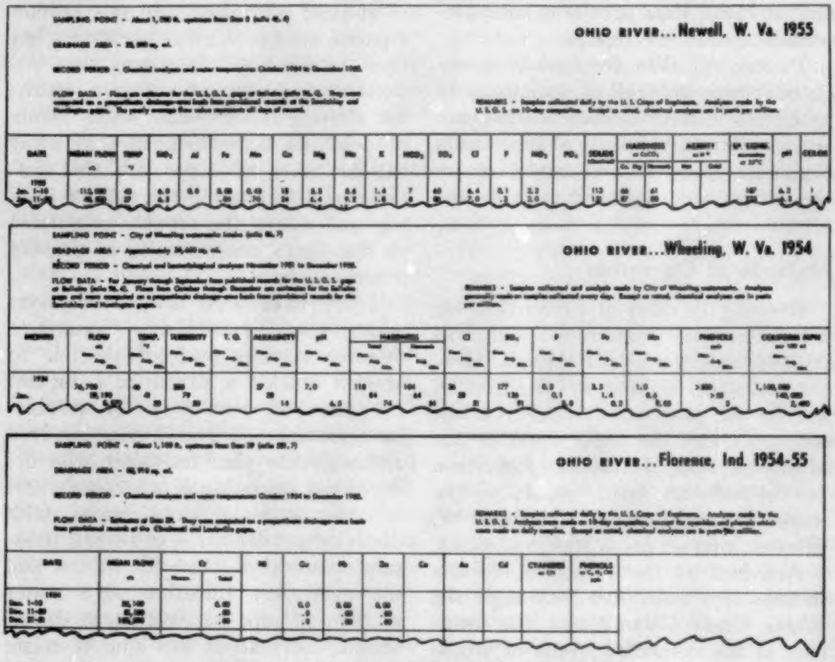

Fig. 1. Samples of Tabulation Sheets

Shown above are three typical forms for recording analytical and flow information from the sampling points along the Ohio River.

In the Columbus laboratory, a measurement of specific conductance is first made on each of the daily samples to provide a quick check on the day-by-day variation, if any, in quality. The

daily samples are then made up into 10-day composites for analysis; the selection of 10-day composite samples for analytical purposes was arrived at after weighing such factors as normal quality variations and cost.

The foregoing routine for delivery and analysis applies to samples collected for mineral analysis. Samples for constituents that have a "die-away" characteristic, such as cyanides and phenols, are collected and analyzed individually by special arrangements. These samples are chemically fixed at the time of collection and then sent by mail to the Columbus laboratory.

All of the USGS analytical determinations, except those for certain Pennsylvania stations, are made under the direction of William L. Lamar, district chemist of the Quality of Water Branch office at Columbus. The Pennsylvania analyses are supplied by N. H. Beamer, USGS district chemist at Philadelphia.

An important contribution to the ORSANCO monitoring program is a special service rendered by the Surface Water Branch of the USGS. Provisional information on river flows is supplied in advance of publication from five Ohio River gaging stations.

Evaluation of Data

Because of the operation of the monitor stations, it is possible to have a thorough collection of data. The compilation and evaluation of the data constitutes a major staff activity.

The analytical and flow information for each station and for each calendar year of record is consolidated on a single tabulation sheet. For this purpose, and after considerable experimentation and discussion with the USGS office at Columbus, a form was devised that is believed to combine several virtues, including legibility, uniformity of reference arrangement, adaptability for reproduction in book form, and production on a readily available office typewriter. The machine chosen for this work was a 20-in. carriage electric typewriter, fitted with a typeface known as Mid Century. This particular type has number characters that are relatively long and thin, and they are easy to read when reduced in size. The care exercised in selecting the machine and typeface for this work resulted from the knowledge that having tabulations of this kind set up in a print shop would be quite costly.

With slight variations, the same tabulation was adapted for compilation of data from the Water Users Committee, and for the special analyses on heavy metals, cyanides, and phenols. Figure 1 shows portions of three of these sheets, each one adapted for a specific location.

The size of the master sheet on which data are recorded is 13¾ × 18½ in. This size lends itself, by proportionate reduction, to reproduction on a standard book or letter size page of 8½ × 11 in.

The tabulation sheet incorporates a calendar year of record instead of that for the water year (October of one year through September of the next year) as used by the USGS. Two reasons dictated this decision. First, a calendar year record in the Ohio Valley district is more likely to include the seasonal low-flow period—a time when quality conditions would be at their worst—whereas a water year record is more likely to subdivide the low-flow period. Second, since the calendar year basis conforms with most other data records, it facilitates processing and use of the quality information. It is generally more convenient to work with calendar year records.

From the basic data as recorded for each station and for each year it becomes possible to make all manner of evaluations. For example, a current study of the staff relates to chloride conditions in the Ohio River. For this purpose, several charts were prepared from which it was possible to make an appraisal of river conditions based on a comprehensive array of facts. Similar studies are being made of hardness and phenols preparatory to making recommendations for control measures.

Not the least important of the facts that are being derived from the basic data is information on the ranges of observed water quality. What the story is for the Ohio River as revealed from the first 4 years of record is shown in Table 1. With such a compilation the

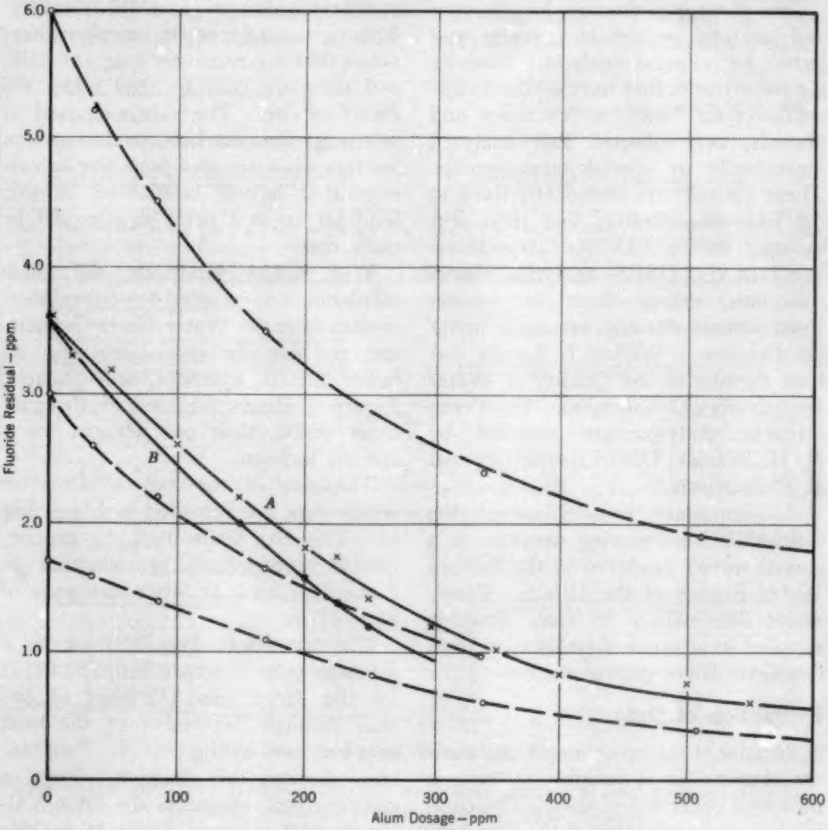

Fig. 2. Flow Duration Patterns for the Ohio River

Curves indicate frequencies of occurrence for flows of various magnitudes at four gaging stations. Arrows indicate the computed average flows. Information was developed by ORSANCO from daily flow records of the USGS.

TABLE 1

Four-Year Summary of Ohio River Water Quality

Item	Monthly-Average Values—*ppm* Min.	Monthly-Average Values—*ppm* Max.	Highest Observed Value *ppm*
Alkalinity (as $CaCO_3$)	2.0	108.0	142.0
Aluminum	0.0	0.3	0.6*
Calcium	20.0	58.0	70.0*
Chloride	6.7	126.0	188.0
Fluoride	0.0	0.8	1.0
Hardness (as $CaCO_3$)			
Total	61.0	271.0	353.0
Noncarbonate	37.0	244.0	342.0
Iron	0.0	2.4	5.0
Magnesium	6.0	17.0	19.0*
Manganese	0.0	1.2	2.0
Nitrate	1.1	7.5	9.6*
Potassium	1.4	4.6	5.8*
Silica	3.0	11.0	19.0*
Sodium	5.0	40.0	45.0*
Solids (dissolved)	137.0	390.0	440.0*
Sulfate (as SO_4)	37.0	362.0	451.0
Item and Unit			
Color—*units*	1.0	29.0	55.0*
Odor—*threshold number*	1.0	71.0	200.0
Phenols (as C_6H_5OH)—*ppb*	1.0	156+	200+
Specific conductance—*µmhos*	220.0	600.0	652.0*
Turbidity—*units*	2.0	576.0	1,500.0

* Based on 10-day composite results.

TABLE 2

Drought Severities Expected at Louisville, Ky.

Drought Severity	Min. Daily Avg *cfs*	Min. Weekly Avg *cfs*	Min. 2-Week Avg *cfs*	Min. Monthly Avg *cfs*
Most probable drought	7,720	11,030	12,160	17,040
Once in 5 years	6,100	8,830	9,220	11,970
Once in 7 years	5,700	7,660	8,490	10,730
Once in 10 years	5,290	6,980	7,740	9,440
Once in 15 years	4,840	6,220	6,910	8,010
Once in 20 years	4,520	5,690	6,340	7,020

commissioners of ORSANCO—and anyone else who wants the facts—can make a quick appraisal of water quality variations as revealed by minimum and maximum monthly averages and in terms of highest observed values. This range sheet will be revised from time to time to incorporate the latest findings. For those who require a more detailed summary on conditions, the yearly maximum, average, and minimum values for various constituents at each station are shown on the individual tabulation sheets.

Hydrographic Data

Chemical and bacteriological data alone are not adequate, of course, for the evaluation of stream conditions. And no pollution control program can be rationally designed without knowledge of the availability of dilution water. Consequently, the diary of a river should incorporate flow data.

Information on stream discharges is meticulously gathered by the USGS. From this wealth of data, studies can be made for the establishment of the flow-variability pattern of a stream, and particularly with regard to minimum flows.

Such studies form part of the river monitor project of ORSANCO. One of these deals with the statistical distribution of all flows of record, and the information is presented in the form of duration curves. These curves establish the patterns of variations of all flows from the lowest to the highest observed. A typical set of curves developed from daily flow records is presented in Fig. 2. This shows the frequency of occurrence for flows of various magnitudes at each of four gaging stations on the Ohio River.

With the stream flow pattern depicted in the form of a duration curve, it is possible to make rapid and accu-

rate estimates of the availability of dilution waters—for example, to answer this question: For what percentage of time is the daily flow of the Ohio River at Sewickley 10,000 cfs or less? Reference to the duration curve will show that this condition has occurred for 30 per cent of the time during the 22 years of record. Further down-river, at Louisville, the appropriate duration curve shows that a flow of 100,000 cfs or less has been experienced for only 4 per cent of the time.

TABLE 3

*Minimum Flows at Louisville, 1934–55**

Year	Day *cfs*	Week *cfs*	Two Weeks *cfs*	Calendar Month *cfs*	Month of Min. Flow
1934	4,900	8,890	10,090	16,200	Jul.
1935	10,600	12,600	13,210	16,800	Oct.
1936	6,150	8,000	8,370	15,010	Sep.
1937	4,090	9,710	10,720	33,420	Sep.
1938	4,600	9,040	10,260	12,270	Oct.
1939	5,200	6,400	7,740	8,590	Sep.
1940	7,200	10,100	10,950	12,930	Oct.
1941	5,360	7,390	8,470	16,190	Oct.
1942	15,200	19,940	27,590	33,720	Sep.
1943	5,300	6,430	6,880	12,450	Oct.
1944	9,120	9,580	10,020	13,440	Aug.
1945	10,700	14,490	16,390	34,310	Jul.
1946	5,500	7,910	9,970	11,050	Sep.
1947	9,730	12,400	14,630	16,050	Oct.
1948	5,780	11,460	12,760	17,650	Sep.
1949	11,000	15,570	17,390	18,950	Oct.
1950	18,000	25,570	29,710	37,000	Aug.
1951	6,000	10,860	11,640	12,350	Oct.
1952	6,420	7,530	8,470	9,760	Oct.
1953	4,220	6,050	6,800	7,320	Oct.
1954	8,600	10,200	11,640	20,480	Sep.
1955	7,130	8,210	8,560	11,740	Sep.

* Computed from daily and monthly discharge records of the USGS.

Information of this kind not only is essential for an appraisal of control requirements, but is invaluable as part of the public record that can be made available to industries and their consulting engineers in locating plant sites.

Duration curves have their limitations, however, as they reveal only the pattern of availability of dilution water based on all flows of record. In pollution control practice, a primary concern is with minimum stream flow and its probability of occurrence. Hence, the ORSANCO hydrographic studies are broadened to include an analysis of drought flow probabilities—specifically, a determination of the minimum daily, weekly, 2-week, and monthly flows.

Results from a typical drought flow analysis tabulation are shown in Tables 2–4. Table 2 shows the flows computed for various drought-severity classifications. Table 3 lists the observed minimum daily flow for each year and with it the computed value for various time intervals. Table 4 shows the flows augmented by the appropriate increment of flow resulting from discharge of water from upstream, multiple-purpose reservoirs operated under direction of the US Corps of Engineers.

The drought-flow studies, which were developed by David Robertson, are based on the statistical theory of extreme values developed by E. J. Gumbel (*1*). Briefly, this called for arranging, in order of descending magnitude, the minimum flow for each year of record. The next step was to compute the percentage of time in years that each flow value was exceeded in magnitude. The flows were then plotted against percentage-of-time occurrence in years on a special graph paper developed by Gumbel; the virtue of this graph paper is such as to secure a linear relation between minimum-flow magnitude and probable frequency of occurrence.

Radiation Monitoring

During the past year, and in generous response to a resolution from the commission requesting the aid of the USPHS, experts from the Robert A. Taft Engineering Center have been conducting a background radiation survey. This important supplement to the ORSANCO quality monitor pro-

gram provides for assay of gross radioactivity at thirteen sampling points on the Ohio River and at 21 locations on tributaries. Additional water sampling is contemplated together with analysis of river muds and stream biota.

On the Ohio River, in the vicinity of the Shippingport, Pa., site of the first nuclear-energy electric generating station, the Atomic Energy Commission has been conducting a preoperational monitoring program. After the start of operations the site monitoring is to be continued by the Duquesne Light Co.

TABLE 4

*Augmented Minimum Flows at Louisville**

Year	Day *cfs*	Week *cfs*	Two Weeks *cfs*	Calendar Month *cfs*
1934	6,310	10,300	11,500	17,610
1935	12,010	14,010	14,620	18,210
1936	7,560	9,410	9,780	16,420
1937	5,500	11,120	12,130	34,830
1938	5,670	10,110	11,330	13,340
1939	6,270	7,470	8,810	9,660
1940	8,270	11,170	12,020	14,000
1941	6,430	8,460	9,540	17,260
1942	16,270	21,010	28,660	34,790
1943	6,200	7,330	7,780	13,350
1944	9,820	10,280	10,720	14,140
1945	11,400	15,190	17,090	35,010
1946	6,200	8,610	10,670	11,750
1947	10,430	13,100	15,330	16,750
1948	5,980	11,660	12,960	17,850
1949	11,200	15,770	17,590	19,150
1950	18,200	25,770	29,910	37,200
1951	6,200	11,060	11,840	12,550
1952	6,620	7,730	8,670	9,960
1953	4,220	6,050	6,800	7,320
1954	8,600	10,200	11,640	20,480
1955	7,130	8,210	8,560	11,740

* Derived from flows shown in Table 3 plus additional flows from reservoirs in upper watershed of Ohio River.

Additional radiation monitoring in the Ohio Valley is being undertaken by several of the states. Two of the states—Ohio and Indiana—have had monitoring programs underway for at least a year.

Costs

The collection and analysis of stream samples to secure data on water quality conditions is generally conceded to be one of the more costly elements of a water pollution control program. It is not unusual for single stream surveys of only a few months' duration and of limited scope to range in cost from $10,000 to more than $75,000. What then, is the cost of conducting a monitor program to provide a continuous and systematic record of stream quality conditions?

Cost information from the ORSANCO monitor program may be helpful to other agencies in making such an evaluation. Table 5 is a summary covering the first 4 years of record. Since the number of stations, as well as the number of months they were in operation, varied from year to year, the comparisons have been prepared on a "station-month" basis.

In developing this program ORSANCO enlisted the volunteer efforts of water utility managers in securing data; the value placed on this effort is equivalent to what is estimated to be the cost if the analytical work had been contracted to a private laboratory. The USGS participation in the program is on a matching basis—thus, for every dollar of work contracted by ORSANCO, the USGS adds a dollar's worth of service.

Depending on the number and type of analyses made, the yearly cost of operating a monitor station can vary from $1,000 (the lowest value placed on data supplied from a Water Users Committee station) to $3,700 (the highest value placed on data from a USGS station). The analyses made at the lowest-cost station include: tur-

bidity, threshold odor, alkalinity, pH, and hardness. Analyses at the highest-cost station include: silica, aluminum, iron, manganese, calcium, magnesium, sodium, potassium, bicarbonate, sulfate, chloride, fluoride, nitrate, dissolved solids, hardness, specific conductance, pH, color, chromium, nickel, copper, lead, zinc, cobalt, arsenic, cadmium, phenols, cyanides, and acidity—a total of 29.

The value placed on each sample for collection and analysis varies from $10 for five constituents (the analyses for which are relatively simple) to $120 for 29 constituents. These figures represent what it could have cost ORSANCO if it had not enlisted the voluntary services of utility managers or engaged in a cooperative program with the USGS and had sought to do the work by some other means. The out-of-pocket cost to ORSANCO for the 4 years of work was $20,000, but the value of the data obtained is estimated at $131,000. Thus, for every dollar invested by ORSANCO the value received in data is estimated to be something more than $6.50.

TABLE 5

Four-Year Summary of Monitor Station Costs

Item	Costs—$				
	1952 (116 station-months)	1953 (141 station-months)	1954 (164 station-months)	1955 (240 station-months)	Total
Direct costs to ORSANCO (including staff time)	3,000	3,000	4,300	10,000	20,300
Volunteer and cooperative-contract services*	20,000	25,000	28,000	38,000	111,000
Totals	23,000	28,000	32,300	48,000	131,300
Avg. cost per station-month	200	200	200	200	

* An estimated value, computed at 6⅔ times the direct costs.

Publication of Data

Although the primary purpose of compiling stream quality and flow data is to provide essential facts to guide the operations of a water pollution control agency, such data obviously are invaluable for many other phases of water resources planning. And such a compilation is of special usefulness to industrial interests and consulting engineers in matters relating to site location.

For these and other reasons, it is desirable to contemplate the publication of the data as a public document. The ORSANCO data have been made available in this form. The first 4 years of record are included in a single volume (*2*). An additional 2 years of record are now being assembled and it is anticipated that these can be made available as looseleaf inserts until such time as a second volume is published.

The cost of printing 1,000 copies of the book, using a photographic process that permitted reproduction directly from the master tabulation sheets, was $1,300. Copies of the book have been distributed gratis to all state and fed-

eral agencies, to some 200 members of committees of the commission, and to municipal libraries in the Ohio Valley district—a total of about 400. For other distribution the commission has established a charge of $2 per copy.

Conclusions

For those agencies contemplating the initiation of a water quality monitor program but who feel that the budget is too limited for such an undertaking, there is this word of optimism: Ingenuity and enthusiasm may uncover resources whose value will more than compensate for the lack of dollars. Also it might be added, a monitor program need not be started on an elaborate scale. The experience of ORSANCO suggests that a modest program can be designed for later elaboration as funds and energy permit. Starting with eleven stations in 1951, the commission has now augmented its monitor network to secure data from 43 locations on the Ohio River and many of its tributaries.

Looking to the future and the possibility of securing more intimate surveillance of river quality variations, for some time there has been planning of what is termed a "robot monitor station" project. Using a portion of a federal grant recently made available through the USPHS under Public Law 660, the commission is now working on this project. The goal, broadly stated, is to investigate the feasibility of adapting analytical instruments for the continuous recording and automatic transmission of river quality data and the development of a self-operating monitor station for this purpose.

Current efforts are directed toward determining the availability and applicability of analytical recording equipment and discussing with certain manufacturers the possibilities of developing equipment not now available. Soon the commission expects to test and operate under field conditions certain combinations of equipment.

It is hoped that the robot monitor project will provide new tools for the evaluation of water quality and prompt detection of pollution control violations. But these stations will not supplant the expert services of the Water Users Committee, the USGS, the Corps of Engineers, and the USPHS in the acquisition of basic data. It is through these agencies and the men that represent them that ORSANCO will continue to rely in perfecting its program of compiling stream quality and flow data.

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Conductometric Control of Coagulant Dosage in Treatment Plants

—*Marvin L. Granstrom and S. David Shearer*—

A contribution to the Journal by Marvin L. Granstrom, Assoc. Prof., Dept. of San. Eng., Univ. of North Carolina, Chapel Hill, N.C., and S. David Shearer of the Robert A. Taft San. Eng. Center, Cincinnati, Ohio.

DURING a series of investigations of the kinetics and mechanisms of coagulation of colloids in water, an attempt was made to use the conductometric technique to make rapid analyses of the rates of coagulation. The procedure was suggested by a paper by Babcock and Knowlton (*1*). In order to become acquainted with their techniques it was decided to try to duplicate the previous work. It was discovered that the results of the two sets of investigations were similar only in certain instances. The areas of difference will be described briefly and certain reappraisals of the conclusions drawn by the previous investigators will be suggested.

The arrangement described consisted of conductivity cells placed in water at a water treatment plant at points before and after the addition of liquid alum—a solution of aluminum sulfate. By appropriate use of an electric integrator and an automatic alum flow rate controller they were able to control the rate of feed of alum according to a predetermined conductivity-difference value.

It was concluded by the previous investigators that the formation of a floc and thus, removal of turbidity, occurred at the dosage of alum which resulted in a sharp increase in the slope of the conductivity curve. They proposed that the slight increase in conductivity during the first (flat) portion of the curve was due to the release of the sulfate ion (SO_4^{--}) from the added aluminum sulfate ($Al_2(SO_4)_3$) and that the second (steep) portion of the curve was due to the release of both the SO_4^{--} and the aluminum ions (Al^{+++}) into the water. They assumed that for the flat portion of the curve the Al^{+++} was consumed in forming the floc. A typical graph of their results is shown in Fig. 1. The equipment was then adjusted so that the alum dosage was slightly greater than that adequate to produce the sharp increase in the conductivity curve. This procedure is working successfully at the water plant at Salem-Beverly, Mass.

Experimental Procedure

After differences in the results of the two sets of investigations were discovered, it was decided to conduct a series of tests to discover the reasons for the differences. For all conductivity measurements a conductivity bridge * which is line operated and capable of supplying a-c frequencies of

* Serfass, Model RCM 15B1, product of Industrial Instruments Inc., Cedar Grove, N.J.

60 and 1,000 cycles per second was used. The latter frequency was used in all experiments. In conjunction with the bridge, a platinum conductivity cell with a cell constant of 1 reciprocal centimeter and a specific resistance of 250–200,000 ohms was used. Water from a constant temperature bath * was circulated through a tank containing the beakers of solution which were under test. During the course of a run, the temperature of the test solution varied no more than ±0.5°C.

All turbidity measurements were made with a turbidimeter † with a 20-cm depth viewing tube. Measurements of pH were made with a line-operated meter ‡ which was also used for making potentiometric titrations for alkalinity. Aluminum determinations were made in accordance with the procedure described in *Standard Methods* (*2*) with a colorimeter.§ The wave length of incident light was set at 525 mμ. Figure 2 shows the equipment used.

The actual procedure for making a run—one run consisted of at least six samples, each containing a different dosage of alum—was as follows:

1. Conductivity, turbidity, alkalinity, and pH measurements were made on all raw samples.

2. Known amounts of alum as $Al_2(SO_2)_3 \cdot nH_2O$ were added to the raw water; the solution was stirred rapidly and conductivity and pH measurements were made of the solution in the reaction vessel.

3. After flocculation and a 2-hr settling period, aluminum and turbidity analyses were made of the supernatant, with all readings taken at a constant temperature.

4. Plots of the recorded data were made (Fig. 3–8).

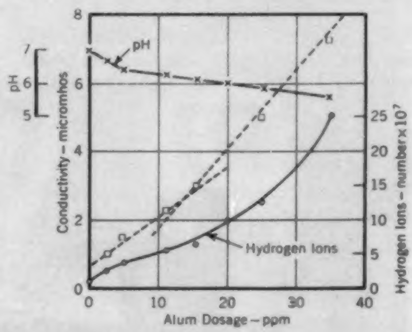

Fig. 1. Typical Conductivity Curve From Salem-Beverly Study

Hydrogen ion (H^+) curve added by authors was derived from pH values given.

Results

It was assumed early in the investigation that the sharp increase in the slope of the conductivity curve occurred primarily as a result of the release of hydrogen ions (H^+) from the water after the addition of the alum. Thus, the alum dosage at which this sharp increase in H^+ occurs—that is, at an appreciable lowering of pH—appears to be determined by the buffering capacity or alkalinity of the water. To test this hypothesis, some 50 different

* Model 9934, product of the A. H. Thomas Co., Philadelphia.

† Product of the Hellige Co., Inc., Garden City, N.Y.

‡ Beckman Model H-2, manufactured by Beckman Instruments, Inc., South Pasadena, Calif.

§ Bausch & Lomb Spectronic 20, Bausch & Lomb Optical Co., Rochester, N.Y.

runs were made using different waters or the same waters in which the alkalinity was adjusted in accordance with experimental design criteria. For this adjustment sodium bicarbonate was used. For each run on a particular water all conditions were held constant except the alum dosage; at least six different alum dosages were used in each run.

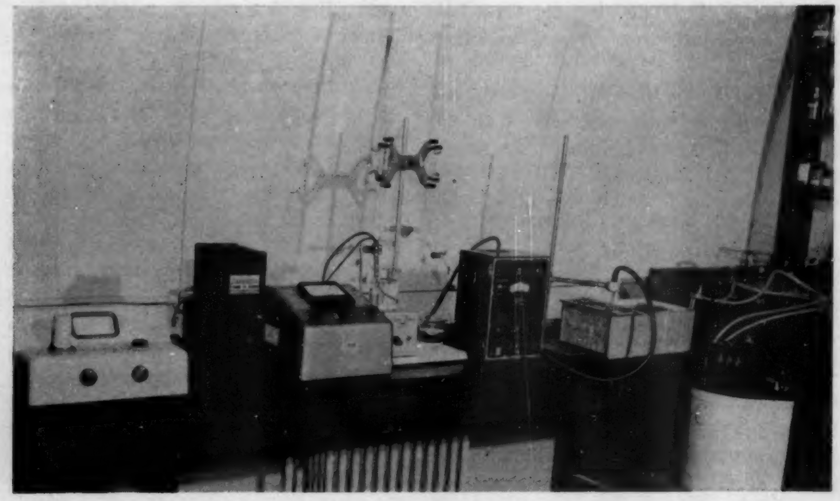

Fig. 2. Equipment Used in Investigation

From left to right, a colorimeter, turbidimeter, pH meter, magnetic stirrer, conductivity bridge and cell, water bath for test solutions, and a constant-temperature water bath.

Following the preparation of the water, each run was conducted in the manner already described. Plots of typical data for three different samples of Chapel Hill, N.C., raw water are shown in Fig. 3, 4 and 5 respectively. The data of Fig. 3 show that essentially all turbidity was removed at dosages of alum much less than that at which the break in the conductivity curve occurred. The alum dosage where the break in the conductivity curve occurred is evidently too high for good turbidity removal. In Fig. 4 the removal of turbidity and the break in the conductivity curve occurred at approximately the same alum dosage and in Fig. 5 the conductivity curve break occurred at an alum dosage lower than that required for good turbidity removal. Of the sixteen runs made with Chapel Hill raw water, the results of seven were similar to those in Fig. 3, four to those of Fig. 4 and five to those shown in Fig. 5. In these sixteen runs, the data show that the location of the sharp break in the conductivity curve bears no definite relationship to turbidity removal. Figure 6 is a plot of the data obtained when sodium bicarbonate was added to increase the alkalinity of the same water used for the

run illustrated in Fig. 4. The results show that the break in the conductivity curve is shifted considerably to the right. This shift was observed whenever alkalinity was increased.

Figures 7 and 8 are results of runs on raw waters from Durham and Burlington, N.C., respectively.

For the water from Durham, the break in the conductivity curve occurred at approximately the optimum alum dosage for turbidity removal. Primary lime was being added to this water by the plant operator and the sample was taken after the addition of the lime. For the water from Burlington, the break in the conductivity curve was at an alum dosage value much greater than the optimum for turbidity removal. On Fig. 3–8 are included the plant alum dosage and alkalinity values on the day of sampling—from which it can be seen that the dosages for good turbidity removal for the experimental runs were about the same as the plant dosages.

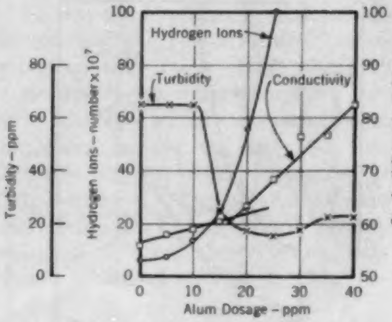

Fig. 4. Effects of Added Alum

This sample was taken Feb. 12, 1957, when plant alum dosage was 29 ppm and alkalinity 13 ppm. Chapel Hill water was used.

Additional runs were made on colloidal systems prepared with Fuller's earth, tannic acid, gelatin, or a dye. The added alkalinity was adjusted at will and again there was no relationship between the alum dosage for good colloidal removal and the alum dosage

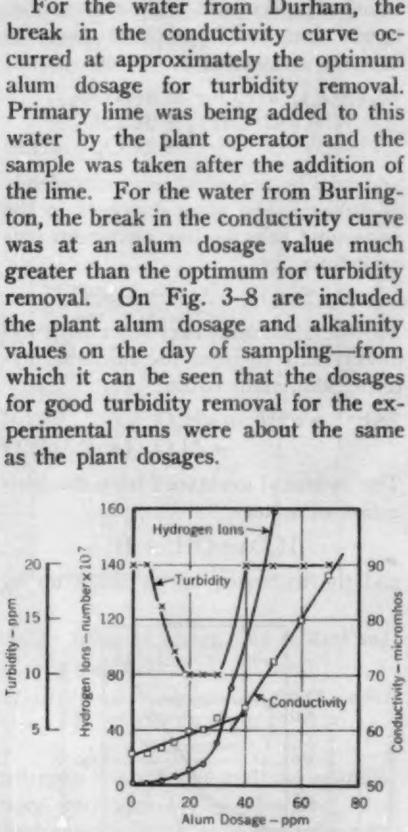

Fig. 3. Effect of Added Alum on Turbidity, Conductivity, and Hydrogen Ion Concentration of Chapel Hill, N.C., Water

Water was sampled Apr. 18, 1957. Plant dosage at the time was 21 ppm alum with an alkalinity of 18 ppm.

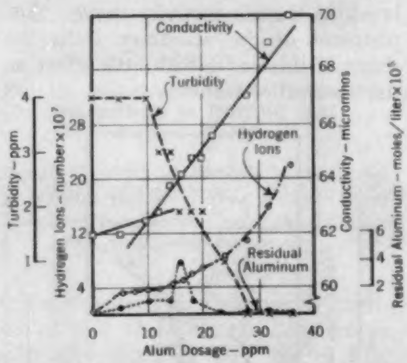

Fig. 5. Effect of Added Alum on Turbidity, Conductivity, Hydrogen Ion Concentration and Residual Aluminum

Chapel Hill water sample taken Dec. 8, 1957. Plant alum dosage 20 ppm; alkalinity, 20 ppm.

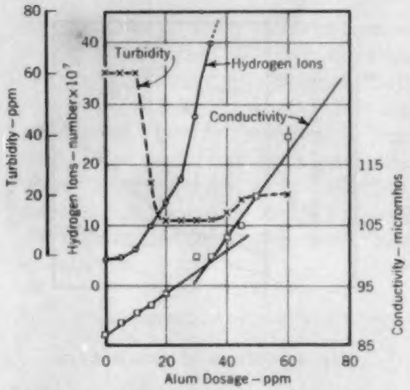

Fig. 6. Effects on Adjusted Water

Chapel Hill water, adjusted with 0.36 me/l sodium bicarbonate. Sample taken Feb. 12, 1957.

at which there was a break in the conductivity curve.

Interpretation

The experimental evidence indicates that the alum dosage necessary for good colloid removal bears no relationship to the dosage at which there is a break in the conductivity curve. Adjustment of the alkalinity shifts the shape of this curve with little effect on the removal of turbidity.

The conductivity-alum dosage relationship indicates that there is some characteristic of the water which changes rapidly at some values of alum dosage. This characteristic is the alkalinity. The conductivity curve is merely an acid-base titration curve in which the alum is the acid and the bicarbonate the base. The reaction between alum and bicarbonate alkalinity in water is commonly written:

$$Al_2(SO_4)_3 \cdot nH_2O + 3Ca(HCO_3)_2 \rightarrow 2Al(OH)_3 + 3CaSO_4 + 6CO_2 + nH_2O$$

and may be described more completely as below. The alum dissolves in the water and releases free aluminum ions as follows:

$$Al_2(SO_4)_3 \rightarrow 2Al^{+++} + 3SO_4^{--}$$

and the aluminum ions react with hydroxyl ions to form the insoluble hydrous aluminum oxide:

$$2Al^{+++} + 6OH^- + nH_2O \rightarrow Al_2O_3 \cdot (n+3)H_2O$$

The hydroxyl ion comes from the ionization of water:

$$H_2O \rightleftarrows OH^- + H^+$$

and the hydrogen ion is taken up by

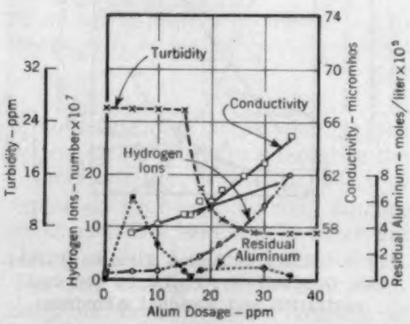

Fig. 7. Effects of Added Alum on Samples of Durham, N.C., Water

Sample taken Dec. 14, 1957, with plant alum dosage 21 ppm; alkalinity, 18 ppm.

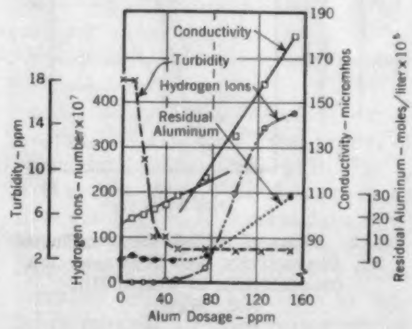

Fig. 8. Effects on Burlington, N.C., Water

Sample taken Dec. 12, 1957. Alum dosage at plant was 32 ppm; alkalinity, 32 ppm.

the alkalinity (HCO_3^-) in the water:

$$HCO_3^- + H^+ \rightleftarrows H_2CO_3$$

Because this last is a reversible reaction, some hydrogen ion will be in solution and the pH of the water is progressively reduced by additions of alum. When the alum dosage stoichiometrically exceeds the alkalinity of the water, however, there is a rapid increase in the concentration of the hydrogen ion present and the conductivity increases quite abruptly; the hydrogen ion is an excellent transport ion. This is the phenomenon observed in this research. There is a possibility that some of the aluminum entered into the colloid which was originally present in the water, but this amount would be small and would not influence the significance of the explanation given above.

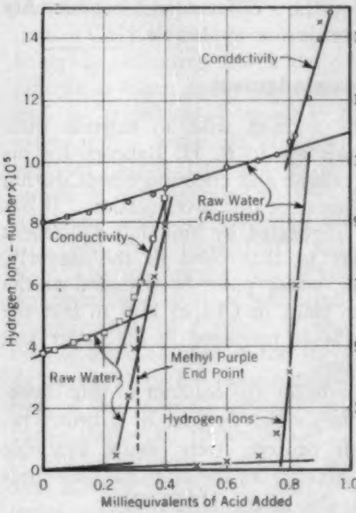

Fig. 9. Effect of Added Acid on Raw and Adjusted Waters

Adjusted raw water had 0.53 me/l $NaHCO_3$ added.

To test the hypothesis, separate potentiometric and conductometric determinations were made of the alkalinity of several of the water samples. Sulfuric acid was added in increments to the water sample and pH and conductivity measurements were made at each increment. The pH values were converted to hydrogen-ion activity which, with conductivity values, was plotted as shown in Fig. 9. The potentiometric and conductometric methods of this acid-base titration are in good agreement, as they are in Fig. 3–8. Actually, Fig. 9 includes a raw water and the same water to which sodium bicarbonate was added. The addition of the sodium bicarbonate shifts the equivalence point to the right in accordance with the amount of base added. It should also be noted

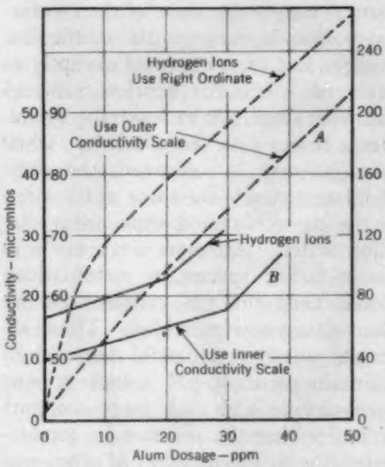

Fig. 10. Relation of Alum Dosage and Conductivity in Distilled and Natural Waters

Dashed lines are for distilled water; solid, for natural water. Curve A *uses outer conductivity scale and Curve* B *uses inner conductivity scale.*

that the increase in alkalinity increases the conductivity of the water.

A further illustration of the relationship between conductivity and alum dosage in water is shown in Fig. 10. In the experiment where alum was added to distilled water, the relationship displays a straight line at the lower dosages and a decrease in the rate of conductivity increase at the higher dosages. In the other experiment, when alum was added to alkaline natural waters the slope of the conductivity line is very gentle at the low dosages and then increases abruptly as expected. The flat portion indicates that the alum (or at least the aluminum) reacts with the alkalinity; when the alkalinity is consumed, the slope of these curves is the same as the slope of the curve obtained when using distilled water. Attempts were made to use a buffer system in water which would keep the pH constant as the alum dosages were added. These attempts were unsuccessful because, to maintain constant pH values, it was necessary to add such large amounts of buffer that the increase in conductivity due to the addition of alum was not satisfactorily measurable.

It is interesting to note from Fig. 9 that the equivalence point of the potentiometric titration for alkalinity in the raw water occurred at a pH of about 5 and the number of milliequivalents of acid added was 0.24. If, however, the sample is titrated to a pH of 4.4—the methyl purple or methyl orange endpoint—the equivalents of acid needed are 0.32—an error of 33 per cent. With increasing alkalinity in the sample, the percentage error is reduced. This is described in *Standard Methods* (*2*).

It is therefore concluded that the location of the break in the conductivity curve is determined by the alkalinity present in the water and bears little or no relationship to the optimum alum dosage for colloid removal. This does not mean that with certain waters—probably those having low dissolved solids—the differential conductivity method is not applicable.

Acknowledgment

The authors wish to express their appreciation to R. H. Babcock for his suggestions and encouragement during the course of this investigation. It has been suggested by him that equipment similar to that used in the Beverly, Mass., water plant be installed at the water plant in Chapel Hill to test the hypotheses proposed in his paper and this one.

Funds for the conduct of this investigation, which is part of a broad research project, were made available by a grant from the National Institutes of Health of the USPHS.

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Control of Algae With Chlorophenyl Dimethyl Urea

Thomas E. Maloney

A contribution to the Journal by Thomas E. Maloney, Biologist, Robert A. Taft San. Eng. Center, USPHS, Cincinnati, Ohio

THE substituted urea compound 3-(*p*-chlorophenyl)-1, 1 dimethylurea (CMU) has been reported as being an effective herbicide. It reputedly is taken up by the root system and is translocated upward to the leaves (*1*). Since its effect on nonvascular plants had not been determined, it was included in preliminary screening tests for potential algicides (*2*). In these tests, CMU displayed good algicidal properties against six representative species of algae at a concentration of 2 ppm. It was completely toxic to four of the species for 21 days and, on the remaining two species, it had a toxic effect during the early stages of incubation. Normal or partial growth, however, was evident after prolonged incubation of the latter species. Exploratory fish toxicity tests revealed that CMU was not toxic to the fathead minnow, *Pimephales promelas,* in the concentrations employed in these preliminary studies.

After completion of the screening studies, full-scale tests were initiated to investigate more fully the effect of this compound upon algae. These studies included both laboratory and field experiments.

Laboratory Tests

The laboratory tests, the culture medium used, the environmental conditions during incubation, and criteria for determining the effectiveness of the test chemical on algal growth were identical with those employed in testing other chemical compounds (*3*).

Fourteen concentrations of CMU, ranging in an arithmetical series from 0.004 to 32.0 ppm, were tested against 33 species of algae. These algae included ten species of blue-green algae, seventeen species of green algae, and six species of diatoms.

Table 1 represents the minimum concentrations of CMU which were effective in preventing normal and partial growth of the algae during incubation periods of 7 and 28 days.

Throughout a 7-day exposure period, a minimum concentration of 0.25 ppm CMU prevented normal growth of all nine species of blue-green algae tested. At the lowest concentration used—0.004 ppm—normal growth of five of these species was controlled. In order to prevent partial growth—that is, to control all visible growth of these species of blue-green algae for 7 days—a concentration of 2.0 ppm was required. Only 0.50 ppm, however, was capable of preventing growth of 7 of these species. When the test cultures were incubated for 28 days, 1.0 ppm CMU prevented normal growth and 2.0 ppm controlled all algal growth. Here a concentration of 0.5 ppm was able to control growth of six of the nine species tested.

As indicated by previous work with other chemical compounds (*3*), the green algae proved to be more resistant than the other classes of algae. A concentration of 0.5 ppm was able to prevent normal growth of the seventeen species of green algae tested during the first 7 days of incubation. To control algal growth completely in this period, a concentration of 8.0 ppm was required. Fourteen of the species, or 82 per cent, were controlled by 1.0

TABLE 1

Minimum Concentrations of CMU Needed to Control Growth of Various Algae

Species Tested	Amount of CMU Used—*ppm*			
	7-Day Exposure		28-Day Exposure	
	Normal Growth	Partial Growth	Normal Growth	Partial Growth
Blue-Green Algae				
Anabaena circinalis	0.004	0.031	0.004	0.250
Calothrix parietina	0.004	0.004	0.031	0.500
Cylindrospermum licheniforme	0.004	0.500	0.062	0.500
Gloeocapsa dimidiata	0.250	0.500	0.250	0.500
Microcystis aeruginosa	0.031	0.250	0.500	0.500
Nostoc muscorum	0.004	1.000	0.250	2.000
Phormidium tenue	0.062	0.500	1.000	1.000
Plectonema nostocorum	0.004	2.000	0.004	0.500
Symploca erecta	0.062	0.500	0.500	1.000
Green Algae				
Ankistrodesmus falcatus	0.031	0.500	0.500	2.000
Ankistrodesmus falcatus v. acicularis	0.004	1.000	0.250	2.000
Chlamydomonas communis	0.500	1.000	1.000	2.000
Chlamydomonas paradoxa	0.125	1.000	1.000	4.000
Chlorella variegata	0.125	8.000	8.000	16.000
Chlorococcum botryoides	0.062	0.250	1.000	2.000
Chlorococcum humicola	0.125	0.500	1.000	2.000
Coccomyxa simplex	0.125	1.000	1.000	4.000
Coelastrum proboscideum	0.250	0.500	0.250	2.000
Gloeocystis grevillei	0.125	4.000	0.500	2.000
Mesotaenium caldariorum	0.004	0.500	0.500	1.000
Oocystis lacustris	0.004	0.250	0.500	1.000
Oocystis marsonii	0.004	0.250	1.000	2.000
Scenedesmus basilensis	0.125	2.000	2.000	4.000
Scenedesmus obliquus	0.008	1.000	0.500	8.000
Sphaerella lacustris	0.125	0.250	0.062	0.500
Stigeoclonium nanum	0.125	1.000	0.062	2.000
Diatoms				
Achnanthes linearis (Strain 1)	0.125	0.250	0.250	0.500
Achnanthes linearis (Strain 2)	0.125	0.500	0.500	2.000
Gomphonema parvulum	0.004	0.500	0.500	1.000
Nitzschia palea (Strain 1)	0.125	0.500	0.500	1.000
Nitzschia palea (Strain 2)	0.062	0.125	0.500	1.000
Nitzschia palea (Strain 3)	0.250	0.250	0.500	0.500

ppm CMU. For control of normal growth for 28 days, one species required a concentration of 8.0 ppm and the remaining sixteen were controlled by 2.0 ppm. This resistant species, *Chlorella variegata,* was completely controlled by 16.0 ppm for a 28-day exposure, and 2 ppm prevented growth of 65 per cent of the other species.

The diatoms were similar to the blue-green algae in their sensitivity to CMU. Normal growth of the seven strains of diatoms tested was controlled by 0.25 ppm for 7 days and 0.5 ppm prevented all growth during this period. During a 28-day exposure, 0.5 ppm allowed only partial algal growth and 2.0 ppm prevented all growth in the seven cultures.

Field Tests

Field tests were carried out at the Ohio State Fish Hatchery at Newtown in three ponds of different sizes. These ponds were designated by the hatchery as Ponds 5, 10, and 20. Because the volume of each pond varied according to the water level, it was calculated immediately prior to treatment. The pH of the pond water ranged from 8 to 10, depending upon the amount of algal growth, and total alkalinity ranged from 125 to 140 ppm.

Pond 5 measured 40 × 160 ft and, at the time of treatment, the calculated volume was 192,000 gal. Before treatment, the bottom was covered by a luxuriant growth of *Spirogyra.* Some surface growth was also present. The first treatment was applied the latter part of April and consisted of a surface spray of 2 ppm of a commercial wettable powder * containing 80 per cent active CMU—providing a concentration of 1.6 ppm CMU.

After 2 days there was no obvious change in the amount of algal growth. After 13 days, however, the bottom of the pond was free of the filamentous algal growth with the exception of a small patch surrounding an inlet spring. Nineteen days after application, the pond was drained to remove the fish. The operator reported that the bottom was clear and required very little raking to remove filamentous algae. It was also evident that the overflow from this pond was having a controlling effect on the algal growth near the point where the overflow entered the adjoining pond.

The pond was immediately refilled and by the middle of September the entire surface was covered by filamentous algal growth consisting chiefly of *Hydrodictyon* with some *Mougeotia.* The same concentration of CMU as previously used was sprayed on the pond and within 2 days there was an obvious reduction in the amount of algae present. Crayfish in the pond migrated to the edges and the fish—smallmouth bass (*Micropterus dolomieu*)—appeared to be agitated. No dead fish were found, however, and the following day they again appeared to be normal. This condition was considered to be due to the low dissolved-oxygen content of the pond water, caused by the oxygen demand of the dead and dying algae, and not to irritation caused by CMU. At this time, two-thirds of the pond was free of algal growth. Six days after the second spraying with CMU, the surface and bottom of the pond were completely clear of algal growth, except for a few dark-brown clumps of dead algae on the bottom.

The tests performed on Pond 10 were conducted to determine the value of CMU in preventing the development of algal growth, rather than its ability

* Telvar W. Monuron, patented by E. I. dupont de Nemours Company, Inc., Wilmington, Del.

to destroy existing growth. Before treatment, the pond, 70 × 140 ft, corresponding to a bottom area of 0.22 acre, was completely drained and cleaned. Filled with water, the pond had an approximate volume of 295,000 gal.

A concentration of 22.3 lb of 80 per cent active CMU (18.2 lb active CMU) per acre was sprayed as evenly as possible over the bottom of the pond before it was refilled. This corresponded to a concentration of 1.6 ppm active CMU when the pond was filled. It was filled immediately after the treatment and remained completely free of filamentous algal growth for 19 days. At the end of this period, a phytoplankton count revealed a concentration of 1,368 cells per milliliter. By the twenty-fourth day after treatment there were a few small clumps of *Cladophora* on the surface and small brown patches of the diatom, *Nitzschia*, were scattered over the mud bottom. The phytoplankton count had reached a concentration of 13,824 cells per milliliter. The amount of filamentous algal growth remained fairly static for 46 days and the phytoplankton count rose to its highest concentration of 125,000 cells per milliliter. *Cladophora* gradually increased and 82 days after treatment approximately one-sixth of the surface area was covered with this growth. The phytoplankton count had decreased to 1,750 cells per milliliter.

Fig. 1. Portions of Treated and Untreated Ponds

Treated pond, at right, is shown 19 days after first treatment with CMU.

At this time, the pond was again treated with 1.6 ppm active CMU. After 4 days, the surface growth of *Cladophora* had completely disappeared and apparently had fallen to the bottom where it was observed to have turned a brown color. The pond remained clear of filamentous algal growth for 20 days after the second treatment

when small clumps of *Cladophora* reappeared. The amount of growth, however, was not appreciable until 34 days after the second treatment when approximately one-third of the surface was covered with *Cladophora*.

During the entire 116-day period following the first treatment there was no evidence that any fish had been destroyed. Although there were no detailed records kept, the hatchery operator reported that the fish in this pond were generally larger than those kept in similar, but untreated ponds. Figure 1 shows the treated pond 19 days after the first treatment with CMU and a portion of an adjacent untreated pond.

Pond 20 measured 162 × 50 ft and had a capacity of approximately 181,000 gal. In mid-May there was a dense growth of *Cladophora* covering two-thirds of the surface area and extending to the bottom. A concentration of 1.6 ppm active CMU was sprayed over the surface in an attempt to destroy the growth of algae. Two days later the filamentous algal growth which had come in direct contact with the spray had begun to turn white. A slight diminishing of the overall amount of growth was evident after 5 days and the white streaks on the surface persisted. In 7 days there was an obvious reduction in the growth which was beginning to turn brown beneath the surface of the water. The condition of the pond improved and the phytoplankton count gradually increased until 18 days after treatment, when the growth of *Cladophora* began to increase and it became obvious that the CMU was no longer controlling it. As in the previous tests there was no indication of death or injury to any fish.

Conclusions

It is conclued from the results of both laboratory and field tests that CMU is an effective algicidal agent. In laboratory tests it was found that 2 ppm prevented growth of all the species of blue-green algae and diatoms tested and the growth of 65 per cent of the green algae. It has the additional advantage of having a low level of toxicity to fish and other aquatic animals. The lethal dose of CMU to a 200-lb man has been estimated at 11.5 oz. (*4*). It is, then, five times less toxic to humans than 2.4 dichlorophenoxyacetic acid (2,4-D). In recent work in connection with the use of CMU to control *Najas*, an aquatic pond weed, it was reported that its influence on the growth of phytoplankton was negligible (*5*). In that work, however, the amount of phytoplankton was measured visually and cell counts were not reported. In addition, the ponds in which the tests were carried out were fertilized regularly. It is possible that some green alga, such as *Chlorella*, which is somewhat resistant to CMU, could have been the dominant form present.

In the field tests, the good results obtained in destruction or prevention of filamentous algal growth are particularly interesting. These algal forms often cause severe problems by blanketing ponds and lakes or by attaching themselves to reservoir walls. They are not only unsightly, but also often choke out other desirable vegetation, prevent normal recreational activities (such as boating, fishing, and swimming), and may impart undesirable tastes and odors to domestic water supplies. The growth often becomes so abundant that the respiration carried on by these algae at night depletes

the water of dissolved oxygen necessary for fish and animal life.

Cladophora appeared to be more resistant to CMU than either *Spirogyra, Mougeotia,* or *Hydrodictyon*—a condition which could have been caused by the presence of *Cladophora* in greater amounts. Spraying of the bottom and sides of a pond with CMU before filling it with water was particularly effective in preventing algal growth. Many of the filamentous algae begin their growth on the bottom of a pond and float to the surface when sufficient oxygen bubbles have accumulated on their filaments. Therefore, treating the bottom of the pond is treatment carried out at the origin of growth and could reasonably be expected to be more effective.

Care should be taken in the application of CMU. It is a selective herbicide at certain concentrations, but at higher concentrations it is a soil sterilant. There is, therefore, the inherent danger of destroying valuable vegetation by misapplication of the compound. After application of CMU to a lagoon in a privately owned park in Ohio, in March 1953, to kill off nuisance water weeds, over 150 trees—some 20–40 in. in diameter—were destroyed (*6*). On this occasion, CMU was applied in concentrations of up to 80 lb per acre, with the substrata of the land surrounding the lagoon permeable to water and conducive to the underground movement of the toxin. Before employing CMU for algal control, consideration should be given to the permeability of the soil on the bottom and sides of the water area, to the drainage in the general area, and to the future use of the treated water. CMU is at present fairly high priced. The relatively small amount required, the duration of its effectiveness, and its low toxicity to fish and man, however, make it a desirable prospect for algae control.

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Fluoride Reduction at La Crosse, Kan.

Russell L. Culp and Howard A. Stoltenberg

A contribution to the Journal by Russell A. Culp, Chief, Water Supply Sec., and Howard A. Stoltenberg, Chief Chemist, Water & Sewage Lab.; both of the Kansas Board of Health, Topeka, Kan.

THIS article describes a method of treating a soft, highly mineralized, well water supply at La Crosse, Kan., to reduce the fluoride content from 3.6 to 1.5 ppm. Basically, the method consists of the application of 225 ppm of alum in increments during rapid mixing and flocculation, followed by settling and rapid sand filtration. The cost for chemicals (alum and lime) at this location is 5.8 cents per thousand gallons. By chlorinating the raw water, iron and manganese are removed in the process. Other methods of fluoride reduction are reviewed as they apply to treatment of this water supply.

The relationships between the fluoride content of drinking waters and dental health are well known. Water supplies may be classified in three general groups according to the amount of fluoride which they contain. More than 1.5 ppm fluoride causes mottling of the tooth enamel and, in high concentrations, discoloration of the teeth; 0.7–1.5 ppm is the optimum range for development of sound teeth; and less than 0.7 ppm causes excessive tooth decay.

In order for these dental effects to be produced the water must be used almost continuously by an individual from birth to 10 years of age. Either the detrimental or the beneficial results are for the most part retained throughout adult life.

The city water supply at La Crosse is obtained from deep wells in the Dakota sandstone, an aquifer which yields an abundant quantity of water. Since the present source of supply was developed in 1922, the water has always contained an excessive amount of fluoride, about 3.6 ppm. Mottling and discoloration of the teeth of persons who spent their childhood in the city have been observed for many years by residents and dentists. During the past 20 years, when the cause of this esthetically objectionable condition became generally known, many La Crosse citizens have obtained drinking water for their children from other sources, often at considerable inconvenience and expense.

City officials recognized that this was not a completely satisfactory solution to the problem, and over the years have attempted to find another source of supply of better chemical quality to replace the present supply or to blend with it. In addition to the high fluoride content, the present supply contains iron and manganese, and although the water is soft, it is high in chlorides, sodium, and total solids. The only alternate source of supply which has been found is located about 5 mi from the city. Water from irrigation wells in this area contains only 0.3 ppm of fluoride, and is of better chemical quality, although it would

require treatment for softening and for iron removal.

Recently, the city requested that the state board of health study methods of fluoride removal which might be used in treating the existing water supply. It was desired to reduce the fluoride content from 3.6 to 1.5–1.0 ppm. This would eliminate the unsightly mottling and staining of teeth, yet would retain the full benefits of the proper amount of fluoride in the water for reducing dental decay.

The laboratory experiments were conducted both in the water and sewage laboratory and in the field, where water samples could be collected from the wells and immediately treated. A total of more than 160 jar tests and 400 chemical determinations were performed. Standard methods of analysis were used throughout the work. To

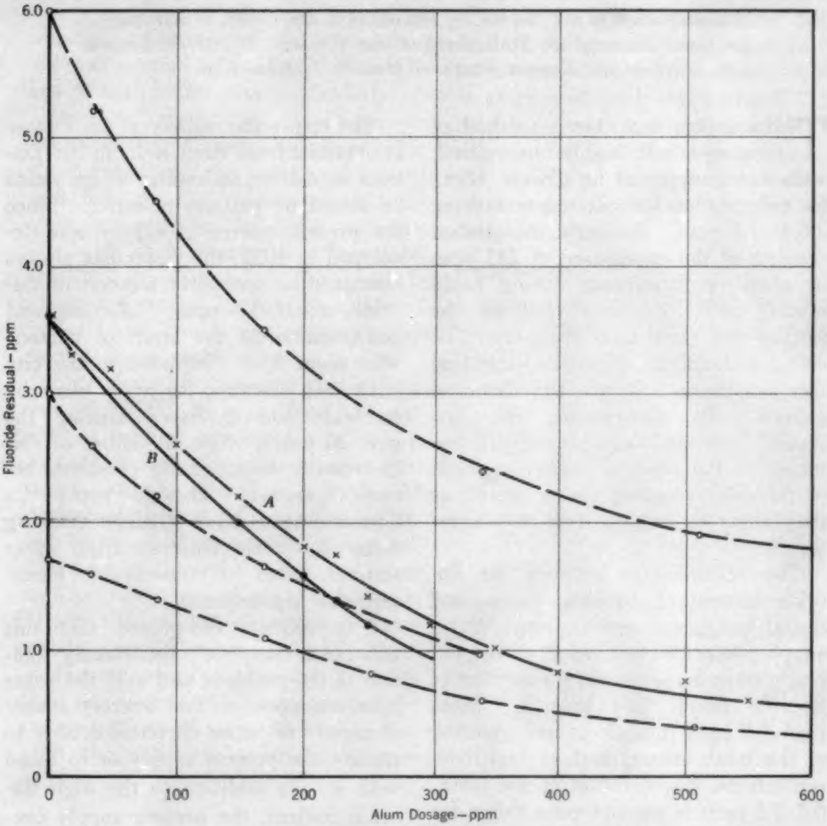

Fig. 1. Fluoride Removal by Alum Flocculation

Curve A *shows the results of experiments with a single point of application (see (Table 1); Curve* B*, increment feeding of alum during mixing (see Table 3). The three dashed curves show results of similar experiments conducted by Scott* (1).

detect possible interference with fluoride determinations, a few key tests were verified by the distillation method. In the jar tests all samples were treated with an excess of chlorine (5 ppm) for iron and manganese removal. All samples were mixed rapidly for 1 min and flocculated for 30 min except as otherwise noted. They were allowed to settle until clear, and then filtered. The raw-water temperature was 62°F.

Alum Flocculation Method

Single point of application. As shown by Table 1 and Fig. 1, fluoride removals are proportional to the alum dosage, although the efficiency appears to decrease as higher dosages are employed. With the single point of application, 250 ppm of alum are required to reduce the fluorides to 1.5 ppm, and about 350 ppm are needed to obtain a residual of 1.0 ppm.

As shown by Fig. 1, these results are comparable to those obtained by Scott in the treatment of Ohio water supplies (*1*). However, Scott used a 7-day contact time with intermittent shaking. Kempf (*2, 3*) and Boruff (*4, 5*) obtained more efficient removals with the use of alum in the treatment of other waters of different chemical composition.

With and without lime. About 15 ppm of lime are required to stabilize the water after treatment. Fluoride removals with and without the addition of lime during rapid mixing were checked as shown in Table 2. Since calcium appeared to interfere slightly with fluoride removal, the lime should be added near the end of the flocculation period. Iron and manganese were removed completely.

Incremental feeding of alum. Three sets of samples were run using this modification. As shown in Table 3 and as illustrated by Fig. 1, approximately 10 per cent less alum is required for the corresponding reduction in fluoride by incremental feeding than with the conventional application at a single point during rapid mixing. This increased efficiency may be the result of better contact between aluminum and fluoride ions during the initial formation of the floc particles and also of more complete coagulation.

One series of three samples was run using the modification of rapid mixing instead of slow. The results and the savings in alum required appear to be very similar to those obtained by incremental feeding during the slow-mixing process. When 225 ppm alum was added to each of three samples during three different time intervals, fluoride residuals were as follows: 3 sec, 1.8 ppm; 1 min, 1.6 ppm; and 2 min, 1.5 ppm. In either modification the fluorides may be reduced to 1.5 ppm by the application of 225 ppm of alum, or to 1.0 ppm by the use of 315 ppm of alum, as compared to 250 ppm and 350 ppm with the conventional single application.

TABLE 1

Single Point of Application of Alum

Alum *ppm*	Fluoride *ppm*
0	3.6
20	3.3
50	3.2
100	2.6
150	2.2
200	1.8
225	1.8
250	1.4
300	1.2
350*	1.0
400*	0.8
500*	0.8
550*	0.6

* Lime was added to maintain a favorable pH range for flocculation. Amounts were 21, 24, 30, and 33 ppm, respectively.

TABLE 2

Alum Flocculation With and Without Lime

Alum *ppm*	pH		Alkalinity*—*ppm*		Fluorides—*ppm*	
	No Lime	15 ppm Lime	No Lime	15 ppm Lime	No Lime	15 ppm Lime
0†	8.0	—	238	—	3.6	—
200	7.0	7.2	162	181	1.8	1.9
250	7.0	7.2	142	156	1.4	1.5
300	6.9	7.1	125	142	1.2	1.2

* As $CaCO_3$.
† Untreated water contained 0.59 ppm iron and 0.09 ppm manganese.

Effect of pH. The fluoride removal is directly dependent upon the efficiency of alum flocculation, which is in turn related to pH. In order to determine the optimum pH range of the La Crosse water for alum flocculation, a series of samples were run with the pH adjusted at the start from 5.0, at intervals of 0.5, to pH 10.5. The pH of the samples after treatment ranged from 5.8 to 10.5.

TABLE 3

Incremental Feeding of Alum During Slow Mixing

Total Alum Dosage *ppm*	Alum—*ppm*					Fluoride* *ppm*
	Time Intervals—*min*					
	0	5	10	15	20	
250	250					1.4
250	125			125		1.4
250	85		85		80	1.4
250	70	60	60		60	1.4
250	50	50	50	50	50	1.4
250	50	40	40	40	40†	1.2
150	75			75		2.2
150	50		50		50	2.0
150	30	30	30	30	30	2.0
200	100			100		1.8
200	80		60		60	1.7
200	40	40	40	40	40	1.6
225‡	225					1.8
225‡	75		75		75	1.6
225‡	60	55	55		50	1.6
225‡	45	45	45	45	45	1.5
225‡	40	37	37	37	37§	1.5

* Untreated water contained 3.6 ppm fluoride.
† An additional 40 ppm alum added at 25 min.
‡ During this set of experiments, residual aluminum was measured; amounts were 0.20, 0.11, 0.10, 0.10, and 0.12 ppm, respectively.
§ An additional 37 ppm alum added at 25 min.

Residual aluminum determinations were made on all samples of this series. Residual aluminum is important for two reasons. It may interfere with the test procedure for fluorides if present in excess of 0.25 ppm (0.25 ppm residual aluminum lowers the apparent fluoride reading by 0.1 ppm). It may also cause deposits to form in water mains and plumbing systems. Solely for experimental purposes, tests were conducted below and above the pH range which would occur in the proposed plant. At these extreme pH values, residual aluminum does become a problem. The residual aluminum is less than 0.25 ppm, however, from pH 6.5 to pH 7.5. Since the best fluoride removal is obtained in this pH range, residual aluminum is definitely not a problem in the proposed plant. Fortunately, in the La Crosse water the required alum dosage produces a pH in the optimum range for fluoride reduction without special adjustment. As previously stated, about 15 ppm of lime is added for stabilization of the treated water.

The data from this experiment are plotted in Fig. 2. Several interesting features are demonstrated. A chart of

flocculation time for alum at various pH values has been superimposed. The close correlation between the efficiency of alum flocculation and fluoride removal is shown. As noted on Table 4, the fluoride values below pH 6.5 and above pH 7.5 may be lower than the true value because of aluminum interference. For pH values between 6.5 and 7.5, however, the area of interest, the interference does not occur and the fluoride readings are true values.

Effect of mixing time. The flocculation time was varied from 5 to 60 minutes on one set of samples without apparent effect on fluoride removal, as all samples showed 1.4 ppm fluoride. The length of time provided for mixing does not appear to be as important as getting immediate contact between the alum and the fluoride ions in the water by adequate stirring and slow addition of the alum.

Multiple flocculation. Based on one test run, two-stage or three-stage flocculation with intermediate settling did not show any improvement over single-stage operation. The fluoride content in all samples was 1.5 ppm. As facilities for this kind of operation would be expensive to construct in the proposed plant, operating costs would have to be reduced substantially to justify their use. As this was not the case, this method was not recommended.

Use of four or more stages probably would produce results similar to those with increment feeding at a like number of points, but the cost of the plant would be excessive.

Use of recirculated sludge. Water samples were treated with one to six volumes of previously precipitated alum sludge, with and without the addition of fresh alum. These trials were disappointing in that no significant reduction in the fluoride content occurred as a result of the sludge contact. Apparently the fluoride enters into chemical combination in the initial formation of the floc or is incorporated in the floc structure near its core, as the once-precipitated floc does not seem to have any further capacity to adsorb the fluoride ion. In 1934, Kempf (*2*) stated that the fluoride was removed by alum as "insoluble hydrated aluminum fluoride."

Coagulant aids. In conjunction with alum, activated silica, bentonite, hydroxyethyl cellulose, and Fuller's earth were used in various dosages as coagulant aids. Coagulation was improved without any significant effect on fluoride removals. As satisfactory coagulation and settling was secured with alum alone, there appears to be no particular advantage in the use of the aids.

TABLE 4
*Effect of pH on Alum Flocculation**

pH†		Fluoride† *ppm*	Residual Aluminum *ppm*
Start	Finish		
5.0	5.8	1.6‡	1.50
5.5	6.5	1.4	0.25
6.0	6.9	1.5	0.20
6.5	7.3	1.8	0.25
7.0	7.7	2.1‡	0.60
7.5	8.0	2.4‡	2.00
8.0	8.2	2.5‡	7.00
8.5	8.5	2.0‡	10.00
9.0	8.8	1.8‡	25.00
9.5	9.3	1.3‡	>25.00
10.0	9.8	1.0‡	>25.00
10.5	10.5	1.2‡	>25.00

* In all samples, 250 ppm alum was added.
† Untreated water had a pH of 8.0 and contained 3.6 ppm fluoride.
‡ Apparent values, subject to possible aluminum interference.

Chemical composition of treated water. The alum flocculation method of treatment satisfactorily reduces the fluoride, iron, manganese, color, and

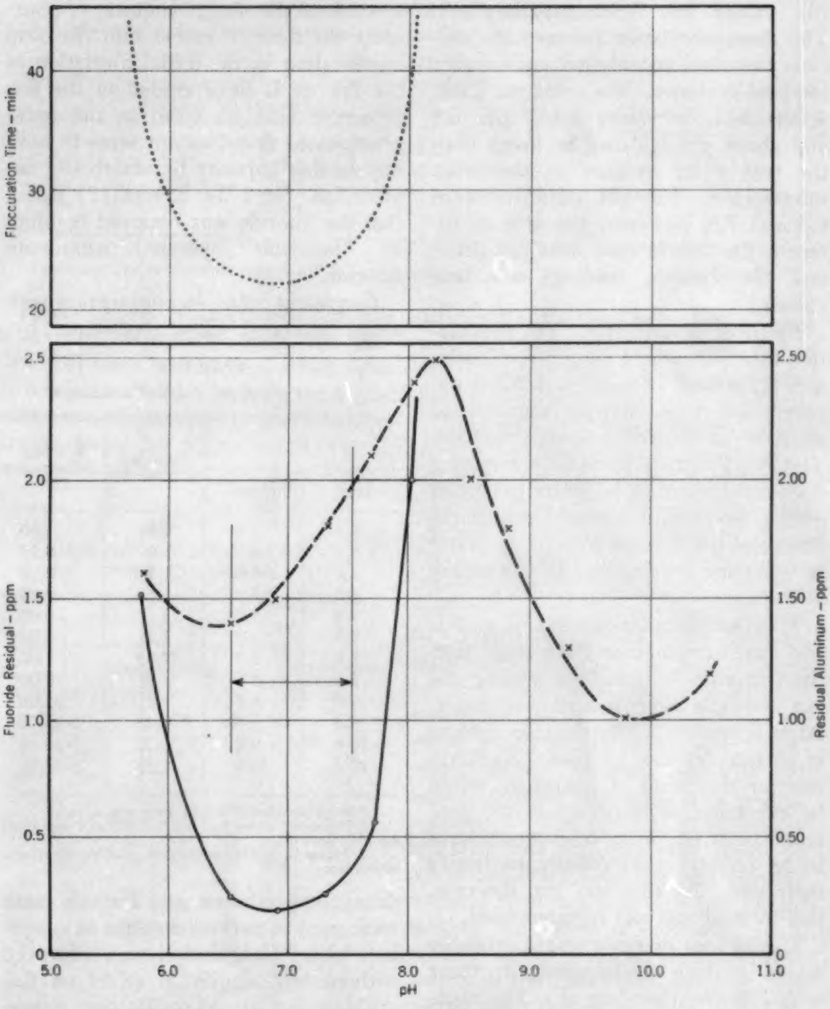

Fig. 2. Effect of pH on Fluoride Removal

Arrows indicate pH area for operation of proposed plant (6.5–7.5), in which area fluoride readings are not subject to possible aluminum interference (see Table 4). Dashes indicate fluoride residual at final pH; dashed curve indicates residual aluminum; and dots indicate flocculation time for alum at various pH values within the area of plant operation. Alum dosages for all tests was 250 ppm.

turbidity to the desired levels. Unfortunately, the finished water is still high in chlorides, sodium, and total solids. Little can be done by this or other practical methods of treatment to remedy this condition, but further improvement may be obtained in the future by development of the alternate source of supply mentioned previously and blending of the two waters. The desired amount of fluoride in the water could then be obtained by regulating the amount of water pumped from the two sources, thus reducing the amount of alum required to only that needed for flocculation. The plant proposed for the present supply could be fully utilized in treating the combined supply from the two sources for softening and removal of iron and manganese.

Magnesium–Lime Softening Method

Fluorides may also be removed along with magnesium in lime softening, the fluoride being adsorbed by the magnesium hydroxide precipitate (*1*). In order to secure the desired amount of fluoride reduction in the La Crosse water it is necessary to add a magnesium compound. Two experiments were run using magnesium sulfate and magnesium oxide. The MgO was dissolved by bubbling CO_2 into water samples to which the MgO had been added. The results are given in Table 5.

Excellent removals were obtained in accordance with the formula (*1*):

$$F_r = F_i - 0.07\ F_i \sqrt{Mg},$$

where F_r is the residual fluoride; F_i, the initial fluoride; and Mg, the magnesium removed. The fluoride content may be reduced to any desired residual by removing the theoretical amounts of magnesium in lime softening. Because of the necessity of adding magnesium to the La Crosse supply, costs would be excessive and this method would not be practical on a plant scale. In general, the successful application of this method is limited to high-magnesium waters requiring softening.

Other Methods

Activated carbon. It has been reported in the literature (*6*) that excellent reductions are obtained with activated carbon at a pH of 3 or below. Application of 50 ppm of activated carbon at the pH of the well water, 8.0, failed to reduce the fluoride content. On a plant scale the cost of reducing the pH, treating with carbon, and then raising the pH would be excessive, so that this was not attempted in the laboratory.

TABLE 5

*Magnesium and Lime Softening Method**

Magnesium† *ppm*	Lime *ppm*	Alum *ppm*	Residual Fluoride *ppm*
Magnesium Oxide			
133	334	60	2.0
133	530	0	1.4
Magnesium Sulfate			
110	560	20	0.8
110	560	40	0.8
110	560	40	1.0‡
80	334	20	0.9
80	334	40	1.2
80	334	40	1.2‡

* Untreated water contained 3.2 ppm fluoride.

† Magnesium oxide as MgO; magnesium sulfate as Mg.

‡ With the addition of 8 ppm activated silica (SiO_2).

Calcium phosphates and ion exchangers. The use of ion exchangers

or any of the various forms of calcium phosphate was rejected on the basis of experience in other places (*7*) and the high costs involved.

Activated alumina. Although this method was not tested in the laboratory, it is probable that the direct chemical costs for fluoride reduction at La Crosse would be lower for the activated alumina method (*7*) than with the alum flocculation method as proposed. Under the prevailing conditions at La Crosse, however, the alum flocculation method has several distinct advantages over the activated alumina process and other methods:

1. The soft water is ideally suited to fluoride reduction by alum flocculation. In higher concentrations, calcium and magnesium would interfere with this method of treatment.
2. The treatment plant would be of simple conventional design in contrast with the more complex plant required for the activated-alumina process and other methods of treatment.
3. The ease of operation is such that no special operating skills are required.
4. Iron and manganese do not interfere with fluoride removal by the alum flocculation method, and they may also be removed in the process by the addition of chlorine to the raw water.
5. The plant could be adapted readily without major changes or additions to treatment of water from other sources which may be developed in the future.
6. Waste disposal is not a problem even though the plant wastes must be discharged to a dry-water course. The alum sludge may be handled satisfactorily in an overflow lagoon.
7. Only conventional water purification chemicals are needed, and there are no strong alkalies or acids to be handled.

On the basis of these advantages, the alum flocculation method was recommended over the activated alumina process.

Proposed Plant

The city has employed a consulting engineer who has reported favorably on the feasibility of constructing and operating a plant to treat the existing water supply by the alum flocculation method with incremental feeding of alum as compared to developing and treating water from the alternate source of supply. The proposed plant would include the following:

1. Addition of 225–315 ppm of alum (Arrangements would be made to feed alum through division boxes to any or all of 3–4 points of application in the rapid-mix basin and to 6 points of addition in the flocculation basin.)
2. Addition of chlorine for iron removal. (Aeration is not recommended, since it is desirable to retain as much of the natural carbon dioxide in the water through the flocculator as possible. This will keep the pH low and increase the efficiency of the alum in fluoride removal.)
3. Rapid mixing of 1 to 2 min, possibly in two units in series
4. Flocculation for 30 min (Incremental feeding probably could best be carried out in an axial flow unit with a high length-to-width ratio.)
5. Addition of 15–20 ppm of lime (The lime should be added in the last section of the flocculation basin.)
6. Settling of 2–4 hr detention (The basin should be equipped with mechanical sludge removal equipment.)
7. Filtration by gravity, rapid sand filters.
8. Sludge ponding (A large overflow pond will probably be the best means to dispose of waste alum sludge.)

The maximum recommended fluoride content as presently set by the Drinking Water Standards (*8*) is 1.5 ppm. The cost of alum (225 ppm) and lime (15 ppm) to reduce the fluoride content of the La Crosse water from 3.6 ppm to 1.5 ppm is 5.8 cents per 1,000 gal. Based on an average annual city use of 150,000 gpd this would amount to about $3,200 per year. In other terms, this would be about $2 per person per year, or an increase of about 60 cents per month in each water bill to cover these operating costs.

From the dental standpoint, the optimum fluoride content for Kansas waters is 1.0 ppm. The cost of alum (315 ppm) and lime (20 ppm) to reduce the fluoride content of the La Crosse water to 1.0 ppm is 8.0 cents per 1,000 gal.

Conclusions

The tests demonstrated that the fluoride content of the La Crosse water may be reduced to any desired level by controlled alum dosages. It was found that the total alum dosage might be reduced 10 per cent by feeding the alum in increments during the mixing and flocculation processes. Since the cost of the alum is the principal expense in the operation of the plant, this is an important modification of the original method. The method is one which should not be overlooked for treatment of other soft, high-fluoride water supplies.

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Aspects of the Sinking of Mexico City and Proposed Countermeasures

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Part 1—General Survey of Problem and Solutions

THE reasons for the sinking of Mexico City have been clarified satisfactorily by measurements, experiments, and calculations carried out during the last decade (*1, 2*). It has been found that the contraction and compaction of the uppermost clay complex—at least 100-ft thick—on which most of the city stands is due not only to surface drainage from the aquifer and to the abstraction of water through numerous shallow wells from the aquifer underlying the colloidal top clays, but mainly to the reduction of pressure caused by pumping from deeper layers. Although most of the 3,000 shallow wells dug by private individuals to a 100–325-ft depth have contributed to the draining of the soil, a far more powerful influence has been exerted on the hydraulic pressure in the uppermost formation by pumping from 200–240 municipal wells, drilled usually to a 325–1,000 ft depth. Abstraction from the individual wells is on the order of 90 cfs and that from the public wells reaches 230 cfs. Originally, the wells in the central part of the city delivered water under artesian pressure. In the course of the last 18 years, this pressure has gradually decreased and led to a lowering of the static level in the center of the city—el 7,346–53 ft—to less than surface levels. Continuation of pumping at the exaggerated present rate of 320 cfs will cause the static level to sink further by about 5 ft per year. This in its turn will produce a continued sinking of the city—the present rate is about 1 ft per year—until the present sinkage of 20–23 ft increases to about 66–72 ft. By that time, a maximum compaction of the soil will have been attained and further withdrawal of water from the underground aquifers, with the concurrent lowering of the water table, would cause no additional sinking of the city.

Because the continued sinking—which is by no means uniform throughout the city—damages existing structures (Fig. 1 and 2) and forces constant changes in the water distribution and sewage systems, the authorities intend to reduce pumping from under the city aquifers at the earliest possible date. It is thought that a decrease in withdrawal from 320 cfs to about 90 cfs will balance natural recharge and abstraction. If all the municipal wells were to cease operating—they now deliver about 230 cfs—a further reduction in pressure would be prevented and the sinking of the city would soon afterward come to a standstill.

The problem, therefore, consists in finding large additional sources of water outside the hydrological limits of the city and in making them available for distribution within a short time (*3*). This task is greatly complicated by three facts:

1. The present supply falls short of present needs.
2. The city is growing rapidly and needs water not only in proportion to the growing population, but also in relation to the rising per capita consumption which derives from a rising standard of living and an expanding industry and agriculture.
3. The distribution network is inadequate at present and requires comprehensive realignment, replacement, and enlargement.

As long as sufficient quantities of new water are not available, pumping from under the city will continue, with all the grave consequences known. The question arises whether further sinking of the soil can be prevented or even delayed with pumping continuing at or near the present rate while the expanded and improved supply system is being developed. Several possible solutions follow.

Deep Aquifers

The subsoil of the Valle de Mexico near the city consists of numerous aquifers separated from each other—either partly or completely—by impervious layers. The potential in the various aquifers differs by as much as 82 ft in some parts of the city, as shown by observations in 1952 for five aquifers between the surface and a 311-ft depth. It may be expected that in the aquifers below 311 ft potential differences will also exist between the different water-bearing layers. These differences prove the partial hydrologic independence of the productive formations.

So far, the private wells have exploited only the upper aquifers to a maximum of 325–500-ft depth and the public wells have penetrated mostly to a 500–1,000-ft depth. An exploratory well is said to have gone as deep as 1,640 ft without encountering bedrock. The results of gravimetric surveys suggest an irregular bottom relief at an estimated depth of 3,300 ft.

It may be assumed, therefore, that water-bearing layers continue in depth intermittently to the bottom of the valley and contain water of usable quality and of a potential largely similar to that in the upper aquifers. It may be further assumed that, under large parts of the city, the deep aquifers are partially or completely separated from each other and from the upper stockworks by impermeable layers. If, under these circumstances, wells were to be drilled to perhaps 2,625 ft to exploit water-bearing complexes between roughly 2,000–2,625 ft (with the higher parts of the boreholes sealed off against the upper aquifers) the reduction of potential from the abstraction of water would communicate itself to the top layer only after a long period. This period might be long enough to provide the breathing spell which the city needs in order to bring water from distant sources.

To evaluate the practicability of the proposal, it would be necessary to determine the physical and mathematical conditions of water abstraction by the use of a multilayer model of the Valley de Mexico to its bottom. Time factors for the reduction of water potentials—assuming pumping at present rates—would have to be calculated, and deep geoelectric surveys accompanied by some test drilling would be needed to

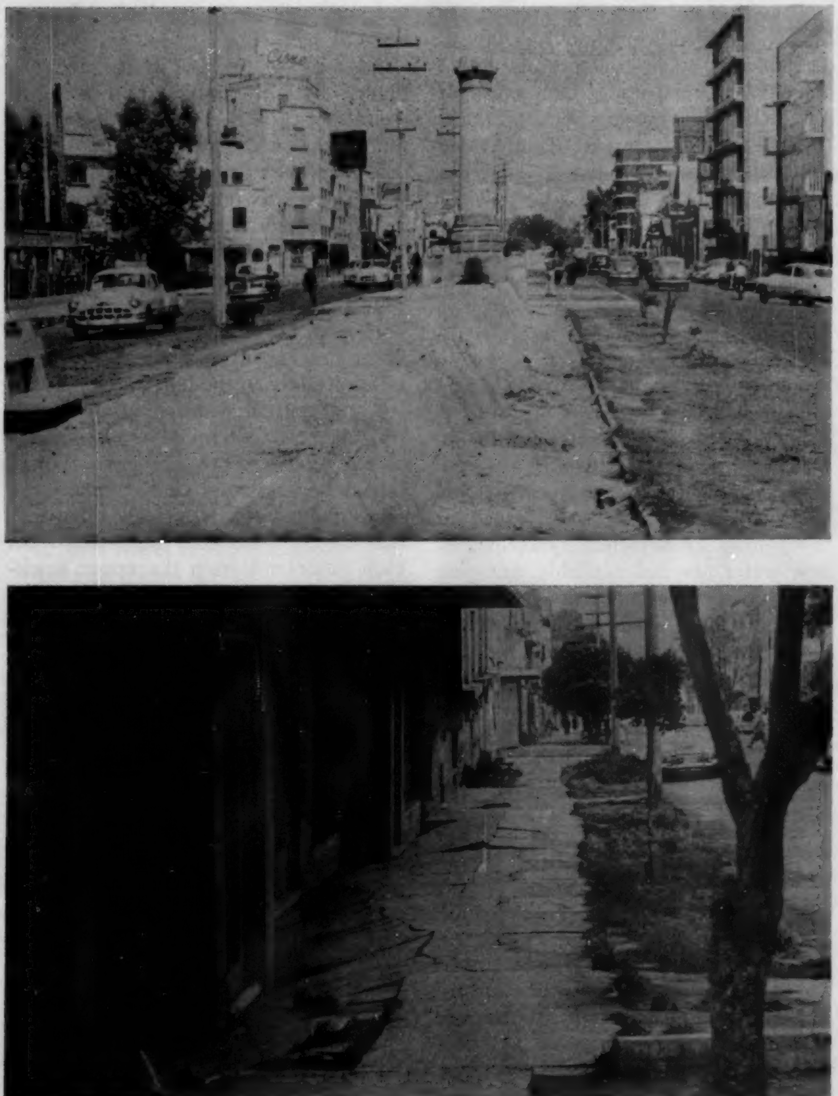

Fig. 1. Effects of Sinking of Soil at Mexico City

Top view is of section of pipeline (left), supported on pillars and originally below street level, which is emerging as ground level at either side falls. Lower photograph shows building unsupported by pillars sinking under own weight more rapidly than sidewalk.

outline the configuration of the valley floor and to determine the total thickness of the aquifer. The economics of replacing the existing shallow wells with deep wells would also have to be calculated.

Selected Recharge

Artificial recharge has for some time been regarded as a means to restore the disturbed balance of water in the drainage area associated with the capital. A number of inverted wells have been constructed—particularly in the Mixcoac section of the city—and have given good results in charging the subsoil with desilted runoff. In order for such operations to be effective, large amounts of water would have to be provided and led underground during the rainy season when surplus is available.

Optimum results in the delaying of sinking could be obtained by charging the first main aquifer—that is, the uppermost porous formation underlying the colloidal sediments of the ancient Texcoco Lake. This means that the position, distance from each other, and depth of inverted wells would have to be adapted to the geohydrological conditions prevailing just under the impermeable top stratum.

Pressurizing With Air

Before 1938, when pumping was started on a large scale in Mexico City, most of the aquifers stood under artesian pressure. The water which freely flowed to the surface from wells, usually increased in quantity with the depth of the borehole. Geohydrologically the situation could be compared to a large vessel having water influx from all sides into a multilayered storage system and bearing in its center a large lid which kept the basin from becoming permanently flooded in its low central portions. Although the pressure under the "lid" has been reduced to such an extent that artesian ground water scarcely exists within or near the city, the dimensions and mechanical conditions of the "vessel" have not

Fig. 2. Exposed Well Casing

Casing is of well drilled in 1923 and is located in part of city where sinking was exceptionally severe.

changed greatly. The lid still exists, although it has sunk by a few feet and its hardness and impermeability have increased slightly through compaction of its components. This means that

there still exists a water container prepared by nature for artesian conditions.

Under these circumstances it might be possible to inject compressed air into the uppermost aquifer and thereby increase the potential of the water still in it.

Closing of City Wells

Although, from an administrative point of view, it would seem easier to close part or most of the municipal wells rather than the private ones, it would be preferable from the hydrological and soil technology viewpoints to start with the elimination of the private wells. Because the private wells are the shallow ones, their elimination would restore at the earliest date pressure in the uppermost aquifer. This is the one which can be damaged most easily by infiltration of harmful bacteria. It should be reserved as a natural filter medium but not used for direct abstraction. In addition, the control over pumping and recharge which the city needs for a long period would be extremely difficult to carry through during the continued operation of a large number of small wells.

Part 2—Details of Use of Artificial Positive Ground Water Levels

The term "artificial water level" is understood to mean a piezometric level of ground water created for and maintained over a limited area. The artificial level, if lower than the water table which existed under natural undisturbed conditions or under previous manipulated conditions, is called a negative artificial level. If raised above the natural or previous position, it may be referred to as a positive artificial level.

General Applications

Negative levels are usually produced in order to drain land to render it more suitable for agricultural or urban purposes. The creation of negative levels is also well known in connection with deep construction works such as cellars, foundations, tunnels, shafts, and mines. The concept of positive artificial levels is relatively new. Objects for its application are numerous, but earth scientists and engineers are only slowly becoming aware of its potentialities. Positive artificial levels may be produced to maintain or attain a certain texture of the ground beneficial for crops, or a uniformity or stability needed for construction. Positive artificial levels may be used to provide aesthetic or recreational projects and may serve transport or water supply functions, as in canals and artificial lakes planned for a natural soil surface. In regional ground water planning—such as is conducted in California and Israel—artificially raised levels near the shore may protect coastal aquifers against the intrusion of sea water, may restrict to a minimum outflow of fresh water into the sea, and may permit a maximum exploitation of the aquifer behind a protective "wall" (Fig. 3). It may provide an artificially raised plateau which is part of a ground water body below (Fig. 4) or a suspended artificial ground water body, "floating" above a natural ground water horizon (Fig. 5). Such an artificial hydrologic structure might be created in answer to a need

for a thin, ground water sheet under a relatively small area. This might be used where the desired level is near the surface relative to the natural ground water horizon which lies at a much greater depth. A factor which would facilitate creation of a suspended ground water body would be low permeability in the zone between the suspended and the genuine horizon. The suspension would in neither instance be complete, except under those conditions where the artificial ground water body rested on a naturally impermeable stratum or on an artificially consolidated layer. Where these separating media do not exist, the hydrological relations between the floating and the natural ground water horizon would be similar to those between a small surface lake and a deep horizon. This means that part of the permeable formation between the two water bodies would be filled with water infiltrating vertically under gravity force, part would flow sidewards to fill empty pores in the surrounding formation, and a third part might be returned to the surface by the forces of capillary action, plant root transfer, and soil temperature differences.

Possible Use at Mexico City

The water department of the Federal District in Mexico (Obras Hidraulicas, D.F.) faces the question of using positive artificial levels as solutions to three problems. The first is developing and maintaining under the whole built-up area of the city a water level which stops further structural damage at the earliest possible date. In selecting an "ideal" water table for this purpose, to be considered are such factors as the past history of the sinking of the city, with effects already produced; the optimum efficiency of drainage; health conditions; and the amelioration of flood damage. Ascertaining the desirable water level can and must be started before the water required for the creation of the new level is at the disposal of the authorities. The desirable water level is not the level which will have been reached when ample alternative water sources have been made available and when pumping from under the city can be curtailed radically, for further sinking and damage will result during the time

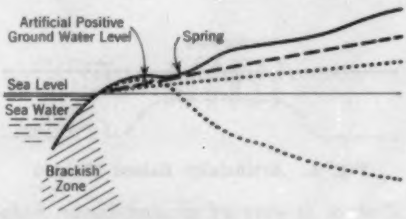

Fig. 3. Artificial Positive Ground Water Level as Barrier to Intrusion of Sea Water

Dashed line represents water table before exploitation of water; upper dotted line, limit of exploitation without barrier; lower dotted line, potential limit with barrier afforded by artificially raised level.

lags between water abstraction, water level reduction, clay compaction, and transfer to structures. It must also be assumed that the quantity of water needed for the reduction of pumpage and for the raising of the level will not be available at one time, but will be at the disposal of the water planners in stages only.

The assumption that a reduction of pumping from 320 cfs to 90 cfs would balance natural recharge with a permanent withdrawal of 90 cfs, may be correct, but it appears doubtful whether it would be desirable to seek a balance be-

tween natural infiltration and abstraction through wells located in the center of the city. Because the soil under the built-up area is almost impervious, natural charging takes place principally at the borders of the basin—where the main well fields should be installed. Optimal withdrawal at the borders of the drainage area will produce a lowering of the ground water table near the foot of the mountain area (Fig. 6). This is likely to restrict the natural recharge under the city's soil to less than the natural volume.

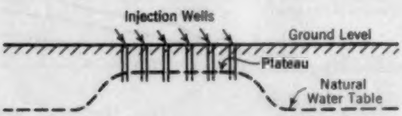

Fig. 4. Artificially Raised Plateau

Plateau is part of ground water body below.

Thus, the object of attaining and retaining a desirable water level under the city should be separated from the object of obtaining optimal quantities of ground water from the border of the Mexico basin. Each requires a different technical solution—the first, regulation by recharge; the latter, limitation only by the yield of the aquifers under proper manipulation and exploitation.

The wisdom of using the resources of the Mexico drainage basin has been confirmed by the fact that water is available in quantities sufficient to cover a major portion of the Federal District's needs. The water is of good quality and will cost less than that derived from other sources. From the security point of view, it is always desirable to abstract water for a large population from nearby and naturally protected ground water at well distributed points in quantities which at least preserve a minimum for emergency demands.

Because the procedures and preparations to delay and ultimately to stop the sinking of Mexico City will not become effective immediately, it may be desirable to protect selected objects in the interim by minor water level manipulations before the larger operation. This would constitute the second possible use of positive artificial levels. The objects to be protected might comprise historic structures or buildings essential to the public welfare. The Xotepingo pumping station provides a suitable example (*4*). The reduction of the water table and its lowering as a result of excessive pumping in the nearby Xotepingo well field, followed by compaction of the upper clay layers and their sinking, has caused and is causing grave damage to the station. A change in the type of foundations—from pillar type to float—might be costly. Isolation of the station by controlled recharge, with an artificial water table raised above the hydrostatic environment would create permanent uniform subsoil and hydraulic conditions. Flexible intake

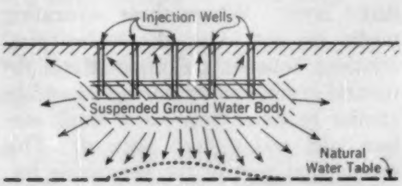

Fig. 5. Suspended Ground Water Body

Arrows show direction of movement of infiltrating water and water moved by capillary action and plant root transfer. Dotted line shows slight raising of water table by infiltration from above.

and output connections could be made to present and future aqueducts leading into and out of the station.

Xochimilco

The third useful application of a local positive artificial level in the Mexico City area would be at Xochimilco, 3–5 mi southeast of the southern border of the built-up area (*5*). The large lake which existed there, is shrinking and the level in the beautiful canals which form the basis of tourist attractions and of intensive garden and flower culture is falling. The drop in the water level is due to drainage and flood prevention measures, to an increase in irrigation around Xochimilco, and to the abstraction of water for the supply of Mexico City—at a rate of 5 cfs—from some of the springs which formerly fed the lake and the canals. The excess of pumping within Mexico City limits may also have influenced adversely the hydrologic regime in the Xochimilco region. The projected increase of pumping from the aquifers underlying the southern hinterland of Xochimilco may accelerate this lowering of the water table.

If abstraction of water at Xochimilco were to stop, the water level would, of course, rise and the local problem of restoring a high level in the earth canals might be solved. The costs in terms of water quantities, however, would be very great and the danger of floods in the southern sectors of the city would increase during rainy years. Evaporation and evapotranspiration losses from a natural, permanently high water table might be 3–6 times greater than the water quantities required for artificially feeding the canals during the dry season. Engineers of the water department at Mexico City have calculated that roughly 1.0–1.3 cfs would be a sufficient rate of artificial recharge to restore Xochimilco Lake to a desirable size and to retain a good level in the canals. The suspension of the lake and the canal levels above the future depressed level of the main aquifer at the border of the Xochimilco zone and below the lake and canal bottom would be aided

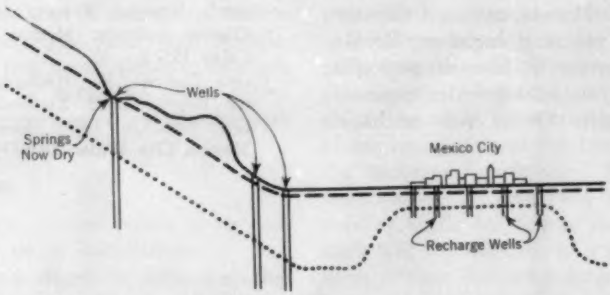

Fig. 6. Artificial Plateau in Relation to Mexico City

Dashed line shows water table before exploitation of water; dotted line, water table upon exploitation.

by low permeability of the collidal soil—which represents a residue of the former large Xochimilco Lake.

Acknowledgment

The description of Mexico City's water problem represents part of the results of an investigation undertaken jointly by the water department of the federal district and the Instituto de Ciencia Aplicada of the University of Mexico. The author gratefully acknowledges the assistance of the following members of the Federal District Water Department: F. Hiriart, director; C. Huerta, assistant director; R. Ochoa, planning engineer; R. Molina, geochemist; F. Mooser, geologist; J. Noriega, consulting water engineer; and J. Sainz Ortiz, soil mechanics engineer.

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Installation, Operation, and Maintenance of Water Pumps

David L. Gallagher

A paper presented on Sep. 25, 1957, at the North Central Section Meeting, Fargo, N.D., by David L. Gallagher, Mgr., Public Works Dept., Worthington Corp., Harrison, N.J.

BEFORE a pump can be operated and maintained it must, of course, first be installed. Although installation may not be of primary interest, there may be occasions when supervision or inspection of installation work is of importance.

Installation

There are several points to be considered in pump installation:

The pump should be located so that it is easily accessible for inspection during operation. At the same time, attention should be given to keeping the suction and discharge piping arrangement as simple as possible. In general, suction piping should be made as short as possible and a suction head provided in order to avoid the necessity of priming. On large pumps there should be ample head room to allow the use of an overhead crane and a supporting structure sufficiently strong to lift the heaviest part of the unit. Motors should not be located in damp places.

After a suitable location has been selected, the foundation must be properly prepared. This may consist of any structure that is sufficiently heavy to afford permanent, rigid support at all points of the bedplate and that can absorb any amount of vibration that may develop from any cause.

Concrete foundations built up from solid ground are most commonly used. An ample allowance for grouting should be made between the underside of the baseplate and the top surface of the concrete foundation. Foundation bolts should be accurately located according to the drawing or template and each bolt surrounded by a pipe sleeve three to four diameters larger than the bolt. After the concrete has been poured, the pipe is held solidly in place although the bolts inside may be moved to conform to the holes in the bedplate. When the unit is mounted on steel work or other structure, it should be set directly over or as near as possible to the supporting beams and walls and be so supported that the base plate cannot be distorted nor the alignment disturbed by any yielding or springing of the structure.

The bedplate is ordinarily grouted-in before piping connections are made. The usual mixture for grouting is composed of one part pure cement and two parts sand, with sufficient water to cause the mixture to flow freely under the bedplate around which a wooden form should be built to contain the grout and provide sufficient head

to insure a flow of the mixture underneath the entire bedplate.

The grouting should be allowed to set for 48 hr, after which the nuts and foundation bolts should be firmly tightened. Horizontal units, in which the pumps and their drivers are shipped mounted on the baseplate, should be supported over a foundation on shims, allowing space for grouting between the bottom of the baseplate and the rough top of the concrete foundation. The coupling bolts should be removed before proceeding with the leveling and the pump and driver shimmed up. Checking of alignment can be accomplished by the use of a straight edge across the top and sides of the coupling while the faces of the coupling half are checked for parallel disposition by means of a thickness gage or set of feelers. When the couplings are perfectly true—on both faces and outside diameters—exact alignment will show the distance between the faces to be the same at all points and the straight edge will lie squarely across the rims at any point.

After all piping has been attached to the pump, alignment must be checked again, because pumps are easily sprung and pulled out of position by the drawing up of bolts and pipe flanges when the flanges are not brought squarely together before the bolts are tightened. Particular care must be taken to support properly the suction and discharge piping so that they do not exert a strain or pull on the pumps. Pipe strains are a common cause of misalignment, hot bearings, worn couplings, and vibration. Aside from misalignment, the greatest number of troubles with centrifugal pumps can be traced to faulty suction line. Too much emphasis cannot be given to the importance of the suction line when installing a pump. The pipe should never be of a size less than that of the pump suction opening and should be as short and direct as possible. Where a long suction line cannot be avoided, the size of the piping should be materially increased. Air pockets or high spots in a pump suction line will invariably cause trouble. Piping must be laid so as to provide a continual rise—without high spots—from the source of supply to the pump. The suction pipe should project into the well or source of supply a sufficient amount to insure that the pipe is adequately submerged when the water is at its lowest level with the pump operating. Large pipes are usually submerged four times their diameter and small pipes require 2–3-ft submergence. Suction pipe should be blanked off and hydrostatically tested for air leakage before starting a pump. Discharge pipe should be installed with a check valve and gate valve near the pump outlet. The check valve provides protection from back flow. On the stuffing boxes, square, soft, asbestos-graphite packing, or its equivalent is recommended. The ordinary grades of flax packing are not recommended for centrifugal pumps with bronze shaft sleeves because rapid wear of sleeves may result. To pack centrifugal pumps properly, exact-length packing rings, cut square, should be inserted separately and pushed as far into the stuffing box as possible by means of a gland. The split of successive packing rings should be rotated 90 deg from those adjacent.

Where water sealing is to be used, the water seal casing should be inserted after two or three rings of packing, making sure the cage is brought directly under the pipe connection. Enough additional packing should be inserted to allow the glands to be

loosely drawn up. Where vertical pumps are being installed, there is, of course, the problem of shafting and installation of the motor on the motor floor level. Care should be taken that the shafting and guide bearings, where used, are properly aligned before being finally bolted. With the pump installed, pipe attached, and motor connections completed, there are still some points to check before attempting to start the pump and put it in operation.

Before the motor itself is connected through couplings or shafting to the pump, it should be tested for rotation to see that it is operating in the same direction as shown by the arrow on the pump casing. The valves on the water seal supply line to the stuffing boxes should then be opened and the pump primed. The pump should not be operated unless it is full of water as there is danger of injuring those internal parts which depend on water for lubrication. It is always recommended that suction and discharge lines be tapped and provided with gage valves for possible testing of pressure in the future. Certain standard units have suction and discharge nozzle flanges drilled and tapped for ¼-in. pipe.

Starting

Final inspection of all parts should be made carefully before starting the pump. It must be possible to revolve the rotors by hand. The bearings should be checked and ball bearings supplied with a good grade of acid-free lubricating grease. Oil-lubricated bearings should be filled level with the overflow. When the pump is full of liquid, the suction valve open, and the unit otherwise ready, a note should be made of the vacuum or static pressure on the suction gage, the driver started, and the pump observed to see that it comes up to speed smoothly. The pump may be operated for a few minutes with the discharge gate valve closed without overheating or damage. Pumps will overheat, however, when operating for any extended time with a closed discharge. The difference between the horsepower input to the pump and the water horsepower output is lost in the unit. This power loss is converted into heat and transferred to the liquid passing through the pump. When operating at shutoff point—that is, with the discharge valve closed—power losses equal the horsepower at shutoff. Because there is no flow of liquid through the pump, all of this horsepower goes into heating the small quantity of liquid in the pump, causing the casing itself to become overheated. In some types of installation, there are bypasses around the pump so that there is a flow until the main discharge gate valve is opened—at which time the bypass valve is closed.

With the larger pump installations, a more extensive tryout than that described above should be made. When this is necessary, the vent valves should be kept open to release pocketed air in the pumping system. Circulation of water prevents the pump from becoming unduly heated. When satisfied that the pump may be put on the line, close the vent valve and open the discharge valve slowly. At this point, if the absolute pressure on the suction head shows a considerable drop compared to the pressure of the pump at rest or if the discharge pressure does not register the moment the rotor is at or near operating speed, the driver should be stopped and all valve openings checked on the suction side. The pump should not be operated until it

is certain that there is a free and ample supply of water and no obstruction in the pipe.

The step-by-step procedure in starting a motor-driven pump can be summarized as follows:

1. Prime the pump after opening the suction gate valves and closing the drains.
2. Open the valve in the cooling-water supply to the bearing.
3. Open the valve in the cooling-water supply where water-cooled stuffing boxes are used.
4. If there is a sealing-liquid supply, open that valve.
5. Start the motor.
6. Open the discharge gate valve slowly.
7. Observe the leakage from the stuffing boxes and adjust the sealing-liquid valves for proper flow to insure packing lubrication.
8. Check mechanical operation of the pump and motor.

In some installations, after the first starting operation the discharge line remains full of liquid and, because this liquid is on the upper side of the discharge check valve, it imposes a head on the pump of sufficient magnitude for starting purposes. It is possible in such circumstances, after the priming or venting, to fill the casing with water and start the pump with both suction and discharge gate valves open.

Operation

Bearings should be carefully watched for signs of overheating. For oil-lubricated bearings, draining is recommended, with washing out and renewal of oil at regular intervals—particularly during the first few months of operation. With grease-lubricated ball bearings, the amount of servicing with additional grease will vary with each unit. For example, on some large motor bearings, once a year is sufficient; but under heavy operating conditions, every 2 or 3 months may be required. The space in the bearing housing should be about one-third full. With the proper grease, if temperature conditions are correct, there will be no softening and consequent escape of lubricant from the bearings. In some installations there are pressure fittings on the pump and there may be lines from the motor floor to the bearing. In these types of installation, it is very easy to get too much lubricant on the ball bearings, which results in their becoming tightly packed with grease. This prevents rolling, and the resulting friction eventually destroys the bearing. It is appreciated that in oil-lubricated bearings, daily attendance and oiling must be maintained. Grease-lubricated bearings, however, need attention only at very infrequent intervals of perhaps 3 months or more.

Normally, a check valve in the discharge line near the pump is used when stopping the pump. With such an arrangement the pump can be shut down by stopping the motor. The valves in the discharge line should be closed first; then those in the suction portion, the cooling-water supply, and any other connections leading to the pump or system. If there is any possibility on high-pressure lines of shock on the discharge line, the discharge gate valve should be shut first, eliminating any shock to the pump. To stop a pump which has been started against the check valve and with the discharge gate valve open—after closing the gate valve and stopping the motor—the valves should be closed in the sealing-liquid line, then those in the bearing-cooling water and stuffing box–cooling water lines, and finally the suction

valve. Drain valves should be left open. Partial draining through the glands will occur when a pump is not operated for some time and, for this reason, a pump should always be primed before starting.

Throttling a pump's suction to reduce its delivery reduces the absolute pressure at the impeller inlet and reduces the capacity by forcing the pump to operate beyond its optimum stage. Such operation is harmful and results in cavitation.

Throttling the suction may be permissible in those special situations when suction pressure exceeds by a good margin the minimum required—such as on the second pump of a series unit. The effect, then, is not to reduce the capacity, but to reduce the total head generated by the series unit. Generally, throttling of the suction in the pump should not be done, because throttling on the discharge gives satisfactory results.

Centrifugal pumps should operate for long periods with practically no attention other than a check that there is always a drip from the glands, and a change of lubricating oil at regular intervals where oil-lubricated bearings are used. Ball bearings should be examined and the lubricant changed at intervals not longer than 1 year. Pumps, even when in good operating condition, will leak without vibration. Bearings operate at a constant temperature which is governed somewhat by the location of the unit. This temperature may be as low as 100°F, but an operating temperature of 150°F or over is not necessarily abnormal. The thrust-bearing temperature may vary with pump capacity, being usually maximum temperature at minimum flow. If a pump does not contain liquid, there will be vibration and overheating as a result of contact between stationary and revolving parts. Cold or comparatively low-temperature liquids should not be admitted to a heated pump quickly as this may result in fracture or distortion of the pump. Vibration is also observed if there is any excessive wear on the pump rotor or in the pump bearing which would result in the pump and motor shaft getting out of alignment.

Usually pumps are driven by motors of the squirrel cage, induction type for constant-speed operation. In those installations where it is desired to vary the capacity of a pump, wound-rotor motors are used. A fairly recent development on large pumps is the use of a magnetic drive. With this type of drive an infinite number of speed steps can be obtained, allowing very close control of the wet-well level by varying the pump speed. Of course, both horizontal and vertical motors are used and, with vertical motors, both hollow and solid shafts.

Causes of Trouble

If the pump is not delivering or is delivering insufficient capacity or insufficient pressure, the reasons may be one or more of the following:

1. Pump not primed
2. Speed low
3. Total delivery head higher than that to which the pump is rated
4. Suction lift too high (Normal limit is 15 ft.)
5. Foreign material in impeller
6. Wrong direction of rotation
7. Excessive air in the water
8. Air leakage in suction pipe or stuffing boxes
9. Insufficient suction pressure for the vapor tension of the liquid (This would apply where hot liquids were handled.)

10. Mechanical defects—such as worn wearing rings, damaged impeller, or defective casing gasket—all of which allow leakage.

Loss of water after starting may be caused by:

1. Air leakage in the suction line
2. Plugged water seal piping
3. Too high suction lift
4. Excessive air or other gas in the water.

Overloading of the driver by the pump may be a result of:

1. Too high speed
2. Total delivery head lower than the rating (The pumps may be pumping too much water "too far out on the curve"—that is, operating between its optimum stage.)
3. Mechanical defects.

Vibration may be caused by:

1. Misalignment
2. Foundation not sufficiently rigid
3. Foreign material in the impeller causing unbalance
4. Mechanical defects such as worn bearings or a bent shaft rubbing a rotating element.

If a pump is operating "in the break"—that is, beyond its optimum operating stage, it will produce a crackling noise which is caused by the excessive suction lift. There are occasions when pumps which operate satisfactorily under full-discharge capacity lose their suction when throttled to a lower rate of flow. This may be caused by air leakage in the pump. At the full-discharge condition the velocities of the liquid in the pump casing are sufficiently high to wash air constantly into the discharge pipe, but at reduced capacity the lower velocities may not wash out the air which accumulates in the casing top and causes the pump to lose its prime.

When pumps are not in use they should be turned over by hand or power at least once a week. All pumps should be opened for examination approximately once a year.

Bearings

Typical bearing construction has the outboard bearing a thrust bearing or thrust and line bearing combined, being a shrink fit on the shaft or sleeves and secured, in addition, with a special nut and lock washer. Outer races are held against end movement between the shoulder and the bearing housing and the spigot on the thrust-bearing cover. The inboard bearing is shrunk on the shaft, for the outer race must have lateral clearance to allow for expansion of the shaft and to prevent binding which causes rough operation. The outer races of both bearings are close sliding fit in their housing and the sleeves on both sides of the inboard bearing are shrunk on the shaft. A pulling device is required for removal of ball bearings. The pulling jaws or fingers must locate behind the shoulder of the inner ball race or its adjacent sleeve—never from the outer race. In addition, it is sometimes necessary to warm the inner race to expand it slightly.

Clearances

In a lateral direction, liberal clearances between rotating and stationary parts allow for slight machining variations and expansion of the casing and rotors when heated. All stationary diaphragms, wearing rings, other parts located in the casing, or other stationary parts, are made only a few thousandths less in diameter than the bore of the casing which is machined with a $\frac{1}{32}$-in. thick gasket between the

flanges. The casing must not bind on these stationary parts when the flange nuts are tightened. All running clearances are 0.001–0.0015 in. per inch of diameter, depending on the actual location of the parts, the material employed, and the bearing span. The lateral clearance between the rotor and stator parts is necessary for several reasons—mechanical and hydraulic—as well as to permit variation of expansion between casing and rotor. The clearance is limited to $\frac{1}{64}$ in. in small pumps and is as much as 0.5 in. on large units.

Pump casings are made from castings. When the pump is built it is often preferable to adjust variations in longitudinal dimensions on the casings by assembly floor adjustments of the rotor in order to preserve the designed lateral clearances and place the impellers correctly with respect to the casing.

Replacement of Rotors

Spare rotors shipped at the time the pump is manufactured have already been fitted and tested in the casing, At other times, parts are made from standard dimensions. It is most important when making field renewals of rotating or stationary parts that all lateral distances be compared with those on the old parts and that the distances where the lateral end movement is affected be exactly duplicated.

Where so fitted, installation should proceed as follows. The assembled rotor should be placed in the lower half of the casing. After checking the total lateral clearance and, with the thrust bearing assembled and the shaft in its proper position, clearance should be rechecked to see that it is suitably divided and the impeller centered in volume. Final adjustments should be made with the shaft nuts. Unnecessary force must not be used to tighten the impeller and shaft sleeve nut because this bends the shaft and destroys the concentricity of the rotor part which, because it operates on close clearances with the stator part, causes rubbing and vibration. When lock screws of the safety type are secured, it should be certain that the shaft is not bent in the process. This can be checked with a dial test indicator. When doing so, indent the thread behind the screw slightly to prevent its backing out.

Annual Inspection

On the annual inspection, the unit should be thoroughly checked—bearings removed, cleaned, and examined for flaws and bearing housings cleaned. Packing should be removed and shaft sleeves checked for wear or, if no shaft sleeves are used, the shaft itself should be checked. Coupling halves should be disconnected and alignment checked. The drain and sealing-water and cooling-water piping should be checked and flushed. Stuffing boxes should be repacked and instrument and metering devices recalibrated.

Renewal of Wearing Rings

New rings should not be added to old, and stuffing box packing, when needed, should be renewed completely. Care should be taken that the seal cage, where used, is opposite the sealing-liquid connection. New packing generally has to be run into it. It is good practice to start and stop the pump several times—for 1- or 2-min intervals—until the stuffing box runs cool with continuous operation. There should always be a slight leakage at the stuffing box for lubrication.

The gland should never be tightened to the point where there is no leakage.

Pumps will continue to operate when the clearance is excessive, but the efficiency and the capacity will be reduced. An increase of 0.01–0.015 in. in diametral clearance warrants renewal of the wearing rings. There are three methods of correcting excessive clearance:

1. After obtaining from the manufacturer a new casing wearing ring, bored under size, the impeller hub wearing ring should be trued up by turning it down in the lathe. This is by far the best method.

2. The worn casing ring surface may be built up by welding or metal spraying so that it can then be bored under size. The impeller hub wearing rings are then trued up as described in the paragraph above.

3. The casing wearing rings may be trued up by boring over size. The impeller hub wearing ring would then be built up and machined to give the correct clearance for the rebored casing ring.

When a new wearing ring is put on an impeller, its surface is often not concentric with the shaft. Therefore, after mounting a new ring, the wearing surface should be checked and machined if necessary. This should be done whether the ring is pressed, bolted, or screwed on to the impeller. The simplest method of repair on large impellers which are pitted or eroded is welding or metal spraying. Special alloy can be used and its cost is justified.

Conclusion

In the final analysis, regardless of what equipment is used, the most significant factor affecting both the operation and maintenance of a pump is the personal element. It is the care taken by pump station personnel which is the most important factor in determining the quality of operation and maintenance.

Converting to Electronic Billing at Hamilton, Ohio

Charles T. Rupert

A paper presented on Sep. 18, 1957, at the Ohio Section Meeting, Cincinnati, Ohio, by Charles T. Rupert, Director of Finance, Hamilton, Ohio.

THE city of Hamilton, Ohio, like a great many other communities, has experienced a rapid growth in both population and area over the past 12 years. This expansion has created many financial problems in the utility department from the production and distribution standpoint. Of course, the billing and collection office has had its share of trouble in keeping pace with added consumers and with a recently added sewer charge. This article describes some of the methods used and some of the problems encountered by the commercial division of the Hamilton utility department in changing its 22,000 gas, electric, water, and sewer accounts from a machine-posting system to an electronic data-processing system.

For over 20 years, the customer accounting in Hamilton has been done on mechanical bookkeeping machines. Billings were made by billing machines after the subtractions and extensions had been entered in the meter book by extension clerks. The computation of the bill was a manual operation, done either by adding machine, calculator, or chart. Errors were inevitable under this system and were unfortunately and too often passed along to the consumer without detection. After the bills were produced, they were sent to the posting clerks for entry on the ledger cards as debits. This was a cycle-billing operation, with one zone billed each day. Cash stubs were turned over to the posting clerks after being sorted into zones by hand and posted to the customer's ledger card daily.

In general, this was a good system. It functioned very well for many years, but it began to become inefficient with the great increase in consumers in the past 7–8 years. During this period of growth, the city was too often faced with a necessity of employing more clerks and of purchasing more equipment, just to keep pace with the additional work load. It soon became obvious that in order to maintain efficiency, some better method of processing the necessary data would have to be found.

Justification

It was at this point that Hamilton officials became interested in the electronic accounting machines. They first requested a survey to determine whether a change in system could be justified. Would the increased efficiency in operation justify the sizable amount of money necessary to install such a system? The first survey was made about 4 years ago and indicated that at that time the city was in a borderline situation. Three years later, it became evident that a sewer rental charge was imminent and that a coal

clause * would be added, which would further complicate the electricity rate. It was then obvious that the consumer accounting had advanced to the stage where electronic data processing would not only be advantageous but economical as well.

The next step was to sell the city manager and the city council on the advantages to be derived from such a change in system. A letter to the city manager pointed out the objectives the change was expected to accomplish: reduction in personnel costs, improvement in speed and accuracy of billing preparation, improvement in effecting rate changes, handling of peak loads without disruption of the normal work routine, and the availability of rate and revenue analyses on either a daily or monthly basis. Discussions with the city manager resulted in his agreement and approval of the proposed program.

The matter was then presented to city council for its approval. The various advantages of the system were readily realized by the council, but one important item was stressed in the presentation. The present personnel would be trained in the new system, rather than hiring experienced people from outside to replace them; in fact, even the supervisor of the tabulating equipment was to be taken from the city ranks. This item really helped to sell the council, as it felt strongly that no employee should lose his job because of the new installation. It was understood that any reduction in personnel, which would be possible through the new system, would be accomplished by not replacing employees who resigned or retired. This might appear to be an item of minor importance, but city officials and city councils are human, and a wholesale discharge program would be most unpopular and would probably be disapproved, regardless of its economy.

* Clause which provides for increase in power rates if the price of coal should rise.

Preparing for Conversion

A changeover is usually thought of as the stoppage of one operation and the beginning of a new one; but in this situation there could be no stoppage of the old operation. It would take time to make the new installation in Hamilton, but under no circumstances could the old operation be discontinued or even allowed to lag. Bills must be computed and mailed on time, at all times. Had it been possible to suspend routine work for even 1 week, the problem would have been tremendously simplified; but this was impossible.

Therefore, programming the new system was probably the most important phase of the new installation. A system similar to one already in successful operation in a private utility in the vicinity was purchased, and the supervisors of the utility department made several visits to see the equipment in operation, to get samples of their forms, and so forth. The equipment company employs men who are specialists in the installation of their equipment and many hours were spent with them, programming every step of the operation in detail. In a changeover such as this, where the company men knew their machines and the utility men knew their system and the results that must be produced, the task amounted to merging the two into a workable team.

Since the meter book is the main record of original entry and is the basis

of all billing information, it was only natural to begin at that point. The following steps were taken before any attempt was made to run the billing register for the first zone to be transferred:

1. All of the accounts in the meter book were renumbered so that no number contained more than four digits.

2. A dual set of account numbers was maintained during conversion, with the old system using its original numbers and the new system using new page numbers which were written in red.

3. After all accounts were in order, the name and address cards were punched for all zones. Once these cards were punched, it was necessary to keep them up to date at all times to maintain accuracy for the time of the actual changeover.

4. The next cards to be punched were the premise cards. A separate card was used for each utility and designated by a different colored stripe for ease in identification. The cards contain the following information: a code number to indicate the particular type of utility; the zone; the meter book and page or account number; the rate class; the serial number of the meter; the constant or multiplier (for an electric meter); the address of the building where the meter is located; and the make of meter and the city's assigned meter number. These cards are used principally for reference and not for any normal machine operation or the printing of bills. They are used to print the necessary information on the new meter book pages, taking much less time than under the old system.

5. When a zone was designated for transfer to the new system, the advance billing card was punched from the meter book about 2 weeks prior to the billing date. These cards contain the code, zone, book, page or account number, rate class, date of previous reading, and previous reading. After the zone was read, the present reading was punched into the advance billing card, and they were ready to calculate.

6. Immediately prior to calculating the new zone for the first time, it was necessary to close out the old system. The ledger cards were balanced, and accounts-receivable cards were prepared from the ledger sheets containing unpaid bills. After the accounts-receivable cards were punched, a trial balance was run to see if they balanced with the old system. During the trial balancing of these cards, the personnel simultaneously punched arrears cards used in the billing of the zone.

Cash Operation

The cash register produces a tape showing the total collections for the day by zones. The stubs are sorted into account numbers by zones and are then turned over to the tabulating department. The stubs are run through the reproducing machine, and a full-sized card is produced, showing the zone, book, account number, and the amount paid. These cards are matched with the accounts-receivable cards and run through the tabulating machine, which prepares a cash journal, listing the detail payments by zones, broken down as to separate utilities. These totals are checked with the cashiers' tapes for balancing purposes.

Trial balances are taken of each zone every 5 days and just before the current billing date. These trial balances are proved by running all of the unpulled or unpaid accounts-receivable cards through the printing tabulator.

Problems of Conversion

Whenever any system is changed, many problems arise. In the first place, it is impossible for anyone to foresee all of the odd little situations that will affect the normal operation of any system. It takes time and patience to discover and eliminate shortcomings, and this can be done only by analyzing and making each adjustment or correction as the situations occur.

Oddly enough, the most difficult problem was selling the system to the personnel. At first, the machines seemed actually to frighten them, and each employee had to be convinced that the same work was going to be accomplished, only with different tools. Cards full of holes rather than words also baffled them. It had to be explained that with the interpreter, they could read punched cards just as easily as any other type of record.

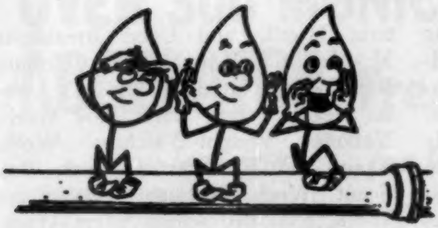
It was quite a problem, too, to divide the employees between the old and the new systems, since it was necessary to operate both at the same time for the changeover period. As was inevitable, the new system ran short handed for a while, and with the resulting overtime for monthly employees, a certain amount of dissatisfaction could have been a problem. Supervisors attempted to arrange for compensatory time off, insofar as possible, but encouragement and the fact that the supervisors were working side by side during the overtime (without hope of compensatory time on their part) helped a lot.

With new machines, more than normal down time is to be expected, which requires more personnel working overtime. More errors are to be expected in a new system, with more checking necessary, and, again, more overtime work.

All in all, the changeover was a tremendous task, requiring a lot of patience, coordination and cooperation to complete the task. Supervisors occasionally wondered if it would be worth the effort.

The job is not complete, but already personnel realize that it has been worth every ounce of the effort. Billing and accounting of water, gas, electric, and sewer charges, with coal clauses, utility tax and discounts, and all the other innumerable complications, will be more efficient, more accurate, more adaptable to expansion, and far more economical. Furthermore, there has been forged a bond of cooperation among all personnel, by their working together through a major upheaval and winning the fight.


In Every Price Bid . . .



Percolation and Runoff

AWWA's wheels—its big wheels, that is—really rolled last Jan. 26–28, and wholly roller during the 3 days of their annual meeting was Prexy Fred Merryfield. As a matter of fact, from the time the roll was called on Sunday noon until the last item of the three-page agenda was cleared 48 hr later, Fred scarcely sat down. What he stood for—the new public information program, the go-ahead on certification, the policy statement on recreational use of reservoirs, AWWA's 31st section, an expanded *Willing Water,* and a host of other items—has already been reported to you in the "Secretary's Letter," but despite the unprecedented busyness, there were a number of nonsecretarian items, too, that kept them rolling—even in the aisles.

Unfortunately for the reportee (though not for the JOURNAL's mailing privileges) it is during the session of the Nominating Committee—from which all but section directors are excluded—that the foremost feats of levity are performed. At any rate, even after the naming of Lauren Grayson veep-nominate thereat, the air of relaxation was noticeable at what is fast becoming an All-Section Dinner. This year it was on bonded bourbon provided by the Kentucky-Tennessee Section, STEAKS from the Nebraska Section, Indian River oranges from Florida, and Delicious apples and Comice pears from the Pacific Northwest that the directors and their wives feasted, and only shipping problems held up the quadruple-old-fashioned glasses from the Canadian (Club) Section until the following day and the Georgia peaches from the Southeast until too late. And after the dinner there was din—the din of some Nevada Section activity from which Mrs. Steve Rankin emerged 74 quarters richer and from which the two who have done the best job of estimating the Dallas Conference * registration and the April 15 membership total will each net a $74 gain.

Back at work, following a spirited discussion of the problem of those dropped for payment of nondues led by Fred Eidsness, the Southern Democrat caucusian, the Board got a roll on again and, with only brief recesses for sleeping and eating, managed to render the "agender" slender enough to take Monday evening off. Then, beginning with a New York Section cocktail party and spreading to various restaurants, shows, and nightclubs,

* See pp. 48–50 P&R for preliminary technical program.

(*Continued on page 36 P&R*)

(Continued from page 35 P&R)

the water works brass (did we hear "brash"?) virtually inundated Broadway. Again more or less unfortunately, the visibility was so poor and the rumors so wild that we can only report all present and presentable when the Board reconvened for its final session on Tuesday morning, as well as right through the New York Section luncheon in their honor—the end of a meeting in which the ratio of historic to hysteric was even greater than usual.

The rolling wheels are spoked, and having spokes move on!

Early to bed went the JOURNAL this month, early to rise, we hope. At any rate, the point of the new production schedule just adopted will be to get the issues into your hands closer to the 20th than the 35th of each month. First victim of the change has been the AWWA annual reports, which the earlier deadlines will necessitate postponing until April. And if this sounds later rather than earlier, such is the press of printing. After all, he who is early to bed doesn't get the morning paper the night before.

Hersey Mfg. Co. has elected Walter A. Hersey chairman of the board. Replacing him as president is Winthrop P. Hersey, who moves up from executive vice-president. The firm is located in Dedham, Mass., where it recently opened its new plant and administrative offices at 250 Elm St.

February, which you have just lived through, but which we yet face, consists of 4 months, 18 weeks, but only 14 days. February, you see, is American Heart Month, Mississippi Recognition Month, Jewish Music Festival Month, and Good Breakfasts Month. It contains National Colorado Beef Week, National Children's Dental Health Week, Rare Book Week, National Weight-Watchers Week, Kraut and Frankfurter Week, Boy Scout Week, National Advertising Week, National Beauty Salon Week, National Crime Prevention Week, National Electrical Week, Negro History Week, National Pimiento Week, Nationally Advertised Brands Week, National Cherry Week, Catholic Book Week, National Engineers' Week, Brotherhood Week, and Pencil Week. Its days are a little less interesting, but run from National Freedom Day on Feb. 1 through Valentine's Day in midmonth back to the freedom of Bachelor's Day on Feb. 28. Hanging over from January, meanwhile, are the last third of Louisiana Yam Supper Month-and-Half and the last half of Time for Hot Chocolate Milk Two Months. All of which is by way of considering a suggestion of Water Works Week to include AWWA's birthday, on Mar. 29. But if hot chocolate rates 2 months, we ought to be able to manage Water Utility Year at least, if not Water Service Century, or, better yet, Moist Millenium!

Graver Tank & Mfg. Co. and its subsidiary, Graver Water Conditioning Co., have been acquired by Union Tank Car Co., Chicago. Since 1949 Graver Tank has produced virtually all the tanks for the tank cars built in the United States by Union Tank, which has a fleet of 57,000 cars serving the petroleum industry and rail shippers. Graver Water Conditioning, with headquarters in New York, will operate as a division of Union Tank, with no basic change in policy.

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(*Continued from page 36 P&R*)

The 'need' in "All the Water You Need, When and Where You Need It!" is one of those words. Written "NEED" it means much less than "need"—much less water, that is, for in our way of life, who needs to get along on the very minimum? If need be, of course, we can sometimes get along on a good bit less just as well or even better. So the General Electric Co. found at its Baltimore plant recently, when, by the use of flow control valves, it was able to reduce water consumption in the cooling jackets of pug mills from 15 to 2 gpm without effect upon the clay cylinders there extruded. Thus, the saving involved was not only 75.4 mil gal in the first year, but $10,315 in water cost as well. Similarly, of course, flow control and pressure control valves on household lines could undoubtedly reduce water use—on the shower head, for instance, or in one of the other time, rather than volume, fixtures. As a matter of fact, according to a Yale University survey, 20 gal—1 gal for drinking, 6 for laundry, 5 for washing, and 8 for waste disposal—is all the average American needs for personal health and cleanliness. Accepting this and the recent revelation that dirty dish water can safely be used to water gardens and lawns, we suddenly begin to understand how a *fountain* pen salesman must feel these days.

Talking about "NEED" rather than "need," about conservation rather than use, is the practice if not the conviction of most water utility men. "Use It Wisely!" we say, "Don't Be a Drip!" "Think Before You Turn the Tap!" "Waste Not, Want Not!" Just imagine Edison Electric Institute in the same position, advertising: "Douse the Glim or Keep It Dim!" "Go by Day or Feel Your Way!" "Save Your Power for a Needy Hour!" "Cook With Gas!" or "Don't Flip That Switch, You S——." There's something wrong with the picture, of course, and what is wrong is water rates. As a matter of fact, if rates were right, we would be promoting conservation without even trying —not by exhortation, but by making water cost enough so that it would behoove the customer to treat it as something of value. But "right rates" aren't the end in view—they're merely the means to "All the Water You Need . . ." or, as the new AWWA public information campaign puts it: *"Improved Water Service Through Water Works Systems, Self-Sustained and Adequate to Meet the Growing Needs of Each Community!"* This our customers NEED!

The NAM board of directors now includes two men from AWWA Associate Member firms. Hubert F. O'Brien, president, A. P. Smith Mfg. Co., was elected director at large of the National Assn. of Manufacturers for a 2-year term, 1957–58, and W. H. Hipsher, executive vice-president, Mueller Co., was elected NAM state director for Illinois for 1958.

O. B. Schier II has taken office as secretary of ASME. Clarence E. Davies, who held the post for 23 years, is now coordinator for the United Engineering Center (see February P&R, p. 48).

Gerald E. Arnold, general superintendent of the Philadelphia Water Dept., has accepted a second term as chairman of the Engineering Manpower Commission for 1958–59.

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(Continued from page 38 P&R)

Engineering enrolment in United States colleges in 1957 was the highest in history—297,000, representing an increase of 7.2 per cent over 1956, compared with a rise of only 4.1 per cent for all college enrolment. The US Office of Education survey from which these figures were taken also reported that engineering freshman enrolment in 1957 was about 79,000—second highest in history; this was a 1.3 per cent increase over 1956, while all college freshman enrolment rose 0.9 per cent. The number of engineering bachelor's degrees in 1956–57 was 18.6 per cent higher than in the previous year, and the number of master's degrees rose 10.8 per cent, but doctorates declined 2.3 per cent.

Engineering enrolment in Canada increased to an even greater extent—12 per cent higher in 1957 than in 1956—according to an Engineering Institute of Canada survey; the number of students was over 14,000. Freshman engineering enrolment was up 13 per cent.

The continued increase in enrolment combined with a slackening in demand for engineers (see February P&R, p. 43) seems to indicate a trend toward a buyer's market. While this is welcome news to employers in the water utility and sanitary fields, optimism should be tempered by the fact that, in terms of starting salaries offered to inexperienced engineering graduates, utility and sanitary engineering is at the bottom of the list of industrial classifications. When the fledglings swarm out of the ivy, they aren't likely to head that way first.

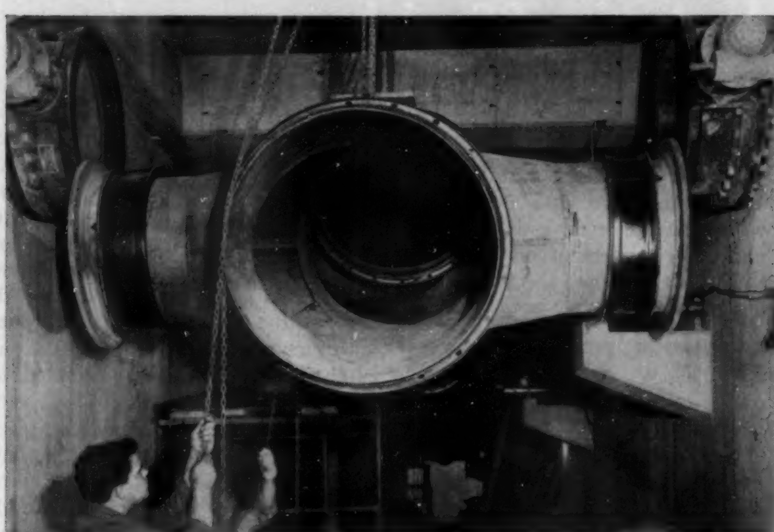
Making a career in sanitary engineering attractive is the aim of a new film, titled "Engineering Your Health," which is being produced for the US Public Health Service for distribution by Apr. 1. In sound and color, the 13.5-min film is directed specifically at college freshmen and sophomores who are science minded. The film should prove equally interesting to alert high school juniors and seniors. A teacher's guide brochure will accompany the reels, which may be borrowed, free of charge, by interested groups from regional offices of the Dept. of Health, Education & Welfare or from Public Inquiries, Public Health Service, Washington 25, D.C. Also available shortly (probably this month) from the same sources will be a vocational guidance booklet now in preparation. Written for high school students, it is titled "Toward a Healthier World: Your Career in Sanitary Engineering."

Elwood L. Bean, chief, Treatment Section, Philadelphia Water Dept., has been named chairman of the newly organized Hydraulic & Sanitary Engineering Div., Philadelphia Section, ASCE. Romeo A. Falciani, design engineer, Albright & Friel, Philadelphia, is vice-chairman.

William T. Ingram and Bernard Newman announce the opening of Newing Labs., Inc., at 260 Islip Ave., Islip, N.Y., serving the sanitary science field.

Thorndike Saville, dean emeritus of New York University's College of Engineering, has been named to head a study on the development of a science and technology center at the University of Florida, Gainesville. The center is planned as a step toward bringing pure and applied science into closer relationship.

(Continued on page 42 P&R)



(*Continued from page 40 P&R*)

A reservoir that floats not on the system, but all through the system area, is being used by the Philadelphia Suburban Water Co. these days to provide an emergency supply to customers in areas where the water has had to be shut off because of breaks in or work on the mains. Actually, the floating is on air in a 2,500-gal tank mounted on a truck and hooked up to a Deming all-bronze centrifugal pump that delivers 30 gpm against a 120-ft head. Residents can come and get it, as shown in the illustration, or, through an alternate connection to approximately 100 ft of hose, the water can be pumped to them.

Just how valuable such equipment can be—or, rather, could have been—was indicated by a recent claim for $35.08 received by the North Muskegon, Mich., city council from an irate resident as the result of a shutdown of one of its water mains during a construction job. The $10 claimed as the cost of cleaning sand from the service line after the shutdown would perhaps have been filed anyway, but the $25.08 for five dinners at a local cafe could certainly have been avoided if a supply had been provided in time to save the dinner that Mrs. Louis Simpson was preparing for her guests when the line was shut down. But more significant than its dollar savings would have been the ire prevention value of an emergency supply, for Mrs. Simpson must have been plenty burned up to have filed that claim.

Get yours now—a reservoir of good will mounted on an ire truck.

A reservoir that flouts, meanwhile, has been the elevated steel tank, proving itself equal to anything up to and including a tornado. It was from suburban Kansas City, Mo., that first proof of its tornado-resistance came, rating a JOURNAL cover picture and story last September. Then in December, the nation's headlines carried the story of the Murphysboro, Ill., twister with pictures of a water tank that succumbed, but it was a gas holder, instead, that had collapsed, the water tank surviving undamaged.

Up to the tornado, meanwhile, have been such indignities as that at Gwinn, Mich., where a year or so ago, the tower was the scene of an apparent hanging, which lured Township Marshal Harry Eberhardt up the 100 ft or more to cut down a lifelike—rather, deathlike—dummy. Or at Lexington, Ky., where, last November, a goat which normally grazes on the University of Kentucky experimental farm was found mountain clinging to the top of a 100-ft water tank, bringing out the fire department to effect a rescue.

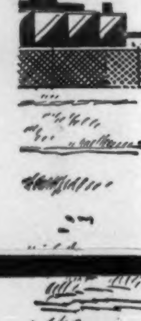
We who prefer to keep our feet on the ground, meanwhile, have a feeling that not only the reservoirs but those that climb them for mischief, duty, or any reason at all are flouting. Ugh!

Finkbeiner, Pettis & Strout, consulting engineers, have moved their Toledo, Ohio, offices from 518 Jefferson Ave. to 2130 Madison Ave.

(*Continued on page 46 P&R*)

Forests Protected . . .

but how about our most precious possession . . . water?



You'll save water—Increase the system Income—with Badger Meters

Forest conservation costs millions — and it's worth it. But the millions of *wasted gallons* of increasingly scarce water *could* cost the future of entire communities. Yet one basic step—replacing flat rates with Badger Meters — sharply reduces waste. Water revenue rises. And because they are Badger Meters, peak meter performance is assured for years.

(*Continued from page 42 P&R*)

Pressure pipe requirements for water and sewage works during the period 1957–75 will total approximately 2.5 billion feet, according to an estimate by the Water & Sewerage Industry & Utilities Div. of BDSA. A recently issued summary of a study made by the division presents a general picture of the recent past, present, and estimated future requirements of pressure pipe for new construction, maintenance, and repair for water supply, sewage disposal, and gas utility distribution. (These estimated requirements do not include the use of pipe for industrial fluids other than water or sewage, or pipe for gas transmission, irrigation, or house services.) The summary is limited to the four principal types of pressure pipe used in the field of water and sewerage works and gas distribution—cast iron, asbestos-cement, reinforced concrete, and steel.

In the field of water and sewage works, the principal segments of total construction involving the use of pressure pipe are the extensions of existing systems to new areas, general improvements to existing systems, construction of entirely new systems, and miscellaneous industrial and federal government military construction. Estimated pressure pipe requirements for gas distribution (661,000,000 ft) include all uses for construction and maintenance and repair in this field.

Copies of "Water and Sewerage Works Pressure Pipe Requirements," Business Service Bul. 213, are available from the Sales & Distribution Branch, Office of Administrative Operations, US Dept. of Commerce, Washington 25, D.C., and from the Commerce Dept. field offices, at 10 cents each.

(*Continued on page 80 P&R*)

How to Choose Waterstop

for optimum performance in concrete joints

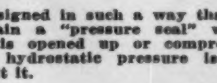
Basic Design And Resilience Most Important Factors

The principal function of waterstop is to keep concrete joints watertight where hydrostatic water pressure is present. To be effective, and to perform its function under widely varying conditions, the waterstop must:

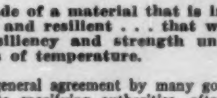
1. **Be designed in such a way that it will maintain a "pressure seal" when the joint is opened up or compressed, or when hydrostatic pressure is exerted against it.**

2. **Be made of a material that is inherently stable and resilient . . . that will retain its resiliency and strength under wide ranges of temperature.**

There is general agreement by many governmental and private specifying authorities, after years of testing and actual installation, that the dumbbell design of waterstop (below) is mechanically superior to any which has been developed to date. The design provides a self-sealing action, because as the concrete contracts and the joint opens up, the outer edges of the dumbbell bulbs become more tightly engaged with the concrete, insuring a tighter seal as the tension increases, due to movement either in the joint or increasing water pressure on one side of the joint. In effect, the greater the longitudinal pull or pressure on one side, the more tightly the dumbbell ends are pulled and squeezed against the concrete. The simpler dumbbell design of the rubber waterstop allows full strength and contact with the concrete surrounding the waterstop. The larger design also provides for maximum strength to resist higher pressures on the web of the waterstop across the joint opening.



6" DUMBBELL TYPE EMBEDDED IN CONCRETE WITH A CONSTRUCTION JOINT



9" HOLLOW BULB TYPE EMBEDDED IN CONCRETE WITH AN EXPANSION JOINT

Rubber and vinyl are the most commonly used waterstop materials. For the majority of applications, rubber is the most satisfactory material. Being a thermosetting material, rubber is more resilient and "live" . . . will maintain a constant pull against the retaining edges (bulbs) as the joint opens up or water pressure increases. Vinyl is a thermoplastic compound and tends to take a "set" after it has been stretched, will float in the joint cavity, and have less resistance to the passage of water. When higher temperatures are present, such as in oil storage tanks, where oil is kept at temperatures around 150°F., the vinyl material, unless specifications are rigidly written, will soften and lose strength, causing a failure of the waterstop.

Field Splicing of Dumbbell Type

Servicised Products Corporation has developed a new Union which provides a simple method of joining the ends of dumbbell waterstops, making it just as fast and easy to field splice rubber and neoprene waterstop as the joining of the polyvinylchloride types.

The Union is made in the same cross-section, and from the same material as the waterstop. It is hollow, except for a solid web at the center. After adhesive is applied to the waterstop ends, they are inserted in the Union and pushed against the centering web. The splice is then clamped together until the adhesive has set. This completes the splice.

Dumbbell type rubber and neoprene waterstop are fully described in a special Waterstop Circular available from Servicised Products Corporation upon request. The Union and an interesting new development, Split Type, are also illustrated and described in the circular. Write for it today.

SERVICISED PRODUCTS CORPORATION

6051 W. 65th Street, Chicago 38, Illinois

Dallas Technical Program

MONDAY, MORNING, Apr. 21

General Session

Presidential Address—Modern Techniques in Water Works Engineering Fred Merryfield
AWWA's Public Information Program .... John H. Murdoch Jr.
AWWA Policy—Recreational Use of Water Storage Reservoirs .... Wendell R. LaDue
AWWA Standards—Shall They Be Enforced? .... Louis R. Howson

MONDAY AFTERNOON, Apr. 21

Water Resources Division

Research in Evaporation Reduction .... Uel Stephens
Conservation and Quality Studies in the Arkansas-Red River Basin .... Kenneth McCall
Discussion .... Paul Fickinger & E. C. Warkentin
Water Works Experiences With the Missouri River Basin Control System ... M. P. Hatcher
Discussion .... W. Van Heuvelen, J. J. Erdei & H. O. Hartung
The Impact of Recreation on the Lower Colorado River .... Robert K. Coote

Water Works Administration Committee—Open Session

TUESDAY MORNING, Apr. 22

Water Resources and Water Works Management Divisions—Joint Session

Ontario's Water Resources Law .... A. E. Berry
Regional Water Supplies—Financing and Cost Allocation .... Richard Hazen
Watershed Development Plans for the Sabine River .... John W. Simmons
Texas Water Problems .... Marvin C. Nichols & T. C. Forrest Jr.

Water Distribution Division

The Value of Cathodic Protection .... George B. McComb
A New Stray-Current Problem .... Hugh L. Hamilton
Fifteen Years' Operating Experience With the Colorado River Aqueduct .. Robert B. Diemer
Plastic Coating and Lining for Steel Pipe .... G. E. Burnett

TUESDAY AFTERNOON, Apr. 22

Water Purification Division

Panel Discussion—Value and Limitations of Chlorine Residuals in Distribution Systems *Led by* H. A. Faber, John R. Baylis, M. P. Crabill, A. E. Griffin & E. J. Umbenhauer
Panel Discussion—Stream Water Quality Monitoring .... *Led by* H. O. Hartung, Edward J. Cleary, Ralph Palange & Lee Streicher
Water Treatment Problems During Low Stream Flow
Part I .... D. F. Metzler, R. L. Culp & H. A. Stoltenberg
Part II .... Richard L. Woodward, S. L. Chang, N. A. Clarke, F. M. Middleton, C. M. Palmer & Graham Walton
Discussion .... C. H. Connell

Water Works Management and Water Distribution Divisions—Joint Session

Making Street Construction Acceptable to the Public .... Burton S. Grant
The Part Played by Water in Modern Fire Control .... M. M. Braidech
Problems in Locating Elevated Tanks .... Clyde E. Williams
Alternate Sources of Power for Water Works Pumping Stations .... Edward Farmer

(*Continued on page 50 P&R*)

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WEDNESDAY MORNING, Apr. 23

Water Purification Division

Effect of Synthetic Detergents Upon Water Softening Economics W. W. Aultman & T. E. Larson
Chemistry of Chlorine Dioxide........M. L. Granstrom
Electrophoretic Studies of Water Coagulation........Janet B. Pilipovich, A. P. Black, A. M. Buswell & F. A. Eidsness
Use of Hydraulically Backwashed Screens for Surface Water........Ernest W. Whitlock & Robert D. Mitchell

Water Works Management Division

Safety in Construction, Plant, and Office.......A. J. Webb, J. E. Hickman & Thomas Allen
Work Simplification for Water Works........W. S. Deniger
Handling Customers and Employees—A Skit.........V. A. Appleyard & Kenneth E. Shull
Organizing the Team........Paul Weir

WEDNESDAY AFTERNOON, Apr. 23

Water Works Administration Committee—Closed Session

Water Resources Division—General Session

Task Group on Ground Water Recharge........J. J. Baffa
Ground Water Reservoirs as Hydrologic Systems........J. G. Ferris
Panel Discussion—Recent Legislation Concerning Water Rights....*Led by* H. E. Jordan, C. H. Bechert, John W. Cramer, A. R. Davis, H. C. Barksdale, Roy W. Morse & Lynn M. Miller

THURSDAY MORNING, Apr. 24

Joint Session With Inter-American Association of Sanitary Engineering

A Review of Water and Sewage Sanitary Engineering Research........H. A. Faber
Report of Water Works Facilities and Construction Throughout the World Fred Merryfield
Development and Operation of Municipal Water Systems in Mexico.....Clicero Villafuerte & Pedro J. Caballero
Sanitary Engineering Educational Programs and Philosophies in Latin America Daniel A. Okun

Management and Distribution Division Workshop

Research—Reduction of Water Main Capacity........T. E. Larson
Research—Developing Protective Coatings for Water Mains........R. F. McCauley
A Study of Domestic Water Use........H. E. Hudson Jr.
Meter Records the Basis for System Planning..Henry J. Graeser, R. B. Raw & A. R. Davis

THURSDAY AFTERNOON, Apr. 24

Water Purification Division Workshop

Manganese Problems........A. E. Griffin
Manganese Research........Floyd I. Brownley Jr.
Iron and Manganese Reduction........E. J. Taggart & V. J. Calise
Planned Research on Filter Washing........W. W. Aultman & J. E. McKee
Some Studies on Filter Washing........Raymond E. Hebert & John R. Baylis
Membrane Filter Task Group........R. L. Woodward & Ray L. Derby
Chlorine Standards and Research........L. L. Hedgepeth
Water-Conditioning Devices........Rolf Eliassen

Water Works Practice Committee—Open and Closed Session

Correspondence

A Tacoman Abroad

To the Editor:

Having completed over 45 years of service in the Tacoma Water Dept., and nearly 39 years of this time as superintendent and chief engineer, I finally was fortunate enough to satisfy a life-long ambition to visit Europe again. After an absence of over 54 years, it was a most interesting experience to see the changes that had taken place in the countries over there. Traveling some 24,000 miles during 3 months through 10 countries brought back many memories of years gone by. The most enjoyable part of the trip was coming back home to our friends and the naturally air-conditioned outdoors in Tacoma.

The tour I joined was conducted by Mrs. Warren Tomlinson, wife of a College of Puget Sound professor, sponsored by Study Abroad, Inc., of New York. As the name of the sponsoring organization implies, this group of 30 ladies and four men was eager to study Europe intensively. In the group were eighteen teachers, including a professor of languages, a professor of zoology, a teacher of mathematics, a botanist, a teacher of dress design, a director of audiovisual education, a student of archeology, a pharmacist-owner of chain drugstores, the owner and publisher of a newspaper, a legislator and writer, a clergyman who provided excellent background on the four basic religions of the world, a food supervisor, a psychoanalyst, and the writer, an expert on public water supplies. The leader of the party, Mrs. Tomlinson, is a lecturer on art and international problems.

Besides the diverse representation of talents within the group itself, we were nearly always accompanied by an expert on the cultural background of the particular country we were visiting. These experts were either professors from one of the universities, artists, teachers, or outstanding people in the business or political field. Contact was also made with the faculty members and student groups of many of the leading universities for the purpose of interesting discussions. These contacts throughout the trip in Europe, and the character of the group itself, made the tour not only extremely interesting, but very much worth while and enjoyable as well.

The one item that I missed most on my menus in Europe, particularly in the southern part, was a good, cool drink of safe water such as we take for granted in our beautiful Northwest. Beer in the southern European countries is flat and tasteless, but was excellent in Switzerland, Germany, and the Scandinavian countries. The same remark could be made about coffee. While good cocktails were always available in England and excellent wine at reasonable prices was obtainable all over the Continent during our trip, the most dependable, though not always the most palatable, drink was coffee.

On the tour we traveled by plane, car, ship, bus, ferry, and foot, and reached the heights of Mt. Pilate in Switzerland by riding a cable car, stretching our necks out over a precipice that was 7,000 ft above the floor of a breathtaking valley. The only water system that I inspected en route was one in a Spanish village—a blindfolded burro plodded around in a circle 40 ft in diameter, turning a pole in the center. This source of power operated a horizontal shaft through a crude bevel-gear arrangement on which was mounted a sprocket wheel with an endless chain of buckets. These buckets moved down into an open well, where

(*Continued on page 54 P&R*)

Correspondence

(*Continued from page 52 P&R*)

the water table stood about 20 ft below ground surface. The filled buckets were raised above the surface and there emptied into a trough that carried the water to a small reservoir. This type of water development served many small farms and communities for irrigation and domestic supplies, though, of course, a modern pump no larger than a small pumpkin could raise more water than this crude contraption.

We found that Spain suffers from an almost complete lack of natural resources. Besides the tourist travel, olive oil and wine seem to be the chief sources of revenue. And practically all the food is saturated with olive oil. One morning I ordered fried eggs in a swank Riviera hotel and they came back dressed, greased, and smothered in olive oil.

The urban areas of Spain seem to display a more modern development than country communities. The benefits of the Marshall Plan are quite apparent in Madrid. Most farming and highway construction we noticed was carried on in a very primitive manner with hand tools and burros. Whereas miles and miles of caves and huts, without windows, electricity, or running water, could be seen in many parts of Spain, the Mediterranean Riviera appeared to be booming with expensive construction equal to the elaborate homes and hotels in such resort areas as Miami Beach or Palm Springs.

In France it is evident that we Americans are not as welcome as elsewhere in Europe. The people seem completely individualistic and appear to be oblivious of what is going on in the world. If this continues, tourist travel to France will fall off.

We also visited the Scandinavian countries, the British Isles, Italy, France, Monaco, Switzerland, and Germany. More construction is going on in Germany now, particularly industrial construction, than anywhere else in Europe. At Oslo about fifteen of Mrs. Kunigk's relatives, and Professor Dagfin Skaar, once an exchange professor at the College of Puget Sound in Tacoma, were on hand to greet and entertain me for 2 days in typical hospitable Norwegian style.

All these countries have their diverse cultural and historical backgrounds and their natural scenic attractions. The privilege of meeting outstanding people in all the countries and traveling with an interesting group of people such as were in our party makes this trip a memorable experience in my life's journey.

The greatest thrill of all, though, was getting back home.

W. A. Kunigk

Tacoma, Wash.
Dec. 15, 1957

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Key: In the reference to the publication in which the abstracted article appears, 39:473 (May '47) indicates volume 39, page 473, issue dated May 1947. If the publication is paged by the issue, 39:5:1 (May '47) indicates volume 39, number 5, page 1, issue dated May 1947. Abbreviations following an abstract indicate that it was taken, by permission, from one of the following periodicals: *BH—Bulletin of Hygiene (Great Britain); CA—Chemical Abstracts; Corr.—Corrosion; IM—Institute of Metals (Great Britain); PHEA—Public Health Engineering Abstracts; SIW—Sewage and Industrial Wastes; WPA—Water Pollution Abstracts (Great Britain).*

HYDRAULICS

Some Observations on Open Channel Flow at Small Reynolds Numbers. L. G. Straub; E. Silberman; & H. C. Nelson. Proc. Am. Soc. Civil Engrs., **82**, EM3, Paper No. 1031 ('56). Results are summarized and discussed of studies on flow in open channels with Reynolds numbers below 4×10^4. Smooth laminar, smooth turbulent, rough laminar, and rough turbulent flows are considered separately, as is transition from laminar to turbulent flow in smooth channels. Results indicate that, at these small Reynolds numbers, smooth channel flow, both laminar and turbulent, is quantitatively similar to smooth pipe flow. Rough channel flow is probably qualitatively similar to rough pipe and rough plate flow, but there is no adequate method available to correlate rough flows in small Reynolds number range. Channel shape is important in laminar flow, but its entire effect may be detd. theoretically. There is only negligibly small channel shape effect in smooth turbulent flow and rather more pronounced effect in rough turbulent flow. Transition usually occurs at slightly higher Reynolds numbers in channels than in pipes, exact effect depending on shape.—*WPA*

A Flow Controller for Open or Closed Conduits. V. L. Streeter. Proc. Am. Soc. Civil Engrs., **82**, HY4, Paper No. 1037 ('56). Principle of flow control is described which is combination of disc moving within profiled throat and nonlinear resistance to support disc against pressure drops. By reversing throat section and measuring head across disc device is converted into flow meter. Flow controller is applicable to both open and closed pipes. Single orifice flow control concepts are outlined briefly, and basic equations are given for design of nonlinear resistance flow control, throat profile, discharge limitations, and flow meter with adjustable sensitivity and range. Exptl. program is outlined and summary of results of readings for 10 settings each as flow controller and flow meter are tabulated. Photograph and diagrams are included.—*WPA*

A New Development in Flow Measurement: the Dall Flow Tube. A. L. Jorissen. Proc. Am. Soc. Civil Engrs., **82**, HY4, Paper No. 1039 ('56). Author describes Dall flow tube, which has been developed for pipe flow metering, and is modification of Venturi tube. Essential differences between these two tubes are outlined; most important are greater compactness and low head loss of Dall flow tube. Exptl. data are given in graphs and tables. Coefficient of discharge for Dall flow tubes can be predicted within 1% when throat to inlet diam. ratios, β, are not more than 0.75. For greater values of β, direct calibration under upstream installation conditions is recommended, unless larger tolerances on coefficient value are acceptable.—*WPA*

An Improved Dilution Method for Flow Measurements. W. A. Cawley & J. W. Woods. Proc. Am. Soc. Civil Engrs., **82**, SA5, Paper No. 1084 ('56). Procedure is described for measuring flow in sewers contg. trade waste waters. An inexpensive technical grade of manganous sulfate is injected into sewer and degree of diln. is detd. quantitatively by flame spectrophotometric anal.—*WPA*

Flow of Fluids. M. Weintraub. Ind. Eng. Chem., **49**:497 ('57). Literature published in 1956 on certain aspects of fluid dynamics is reviewed. Subjects reviewed are: single-phase flow, including simple channels, flow through equip., surge flow, non-Newtonian fluids, jet mixing and boundary

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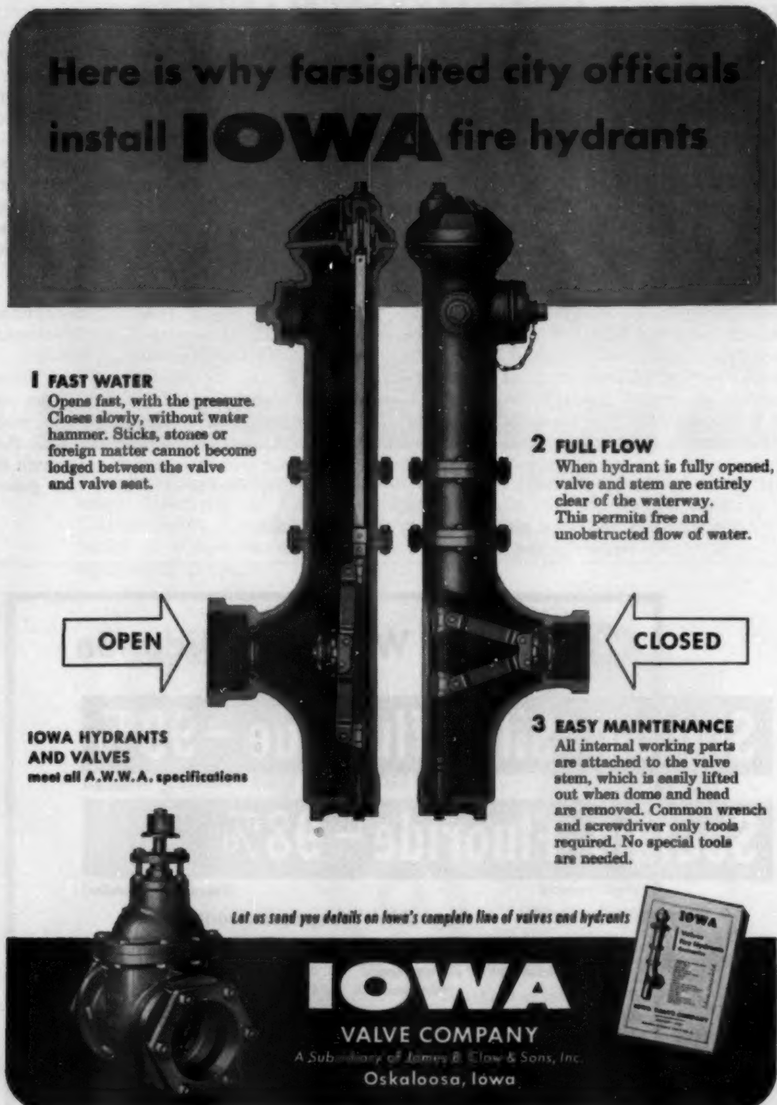
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layers; flow through porous media; multiphase flow including solid-gas, solid-liquid, and gas-liquid systems; and mechanical design including piping and accessories, rotating machinery, and cavitation. A list of 141 references is appended.—*WPA*

The Application of Sediment-Transport Mechanics to Stable-Channel Design. E. M. Laursen. Proc. Am. Soc. Civil Engrs., **82**, HY4, Paper No. 1034 ('56). Author outlines 3 relationships which are needed for design of stable alluvial channel. These are flow equation, sediment-transport equation, and bank-erosion criterion. He assesses role of sediment transport in each relationship. Tables are given showing similarity of many bed-load and "Kennedy" formulae for calcn. of sediment transport. General method of application of these formulae to design of channels is illustrated, and it is suggested that formulae should be used as scaling relationships between different channels. List of references is appended. —*WPA*

Mechanics of Sediment-Ripple Formation. H.-K. Liu. Proc. Am. Soc. Civil Engrs., **83**, HY2, Paper No. 1197 ('57). Author reviews literature on mechanics of sediment ripple formation and shows that ripples are caused primarily by instability of zone of high veloc. gradient at surface of sediment-laden bottom. For practical application, exptl. criterion is given to predict formation of sediment ripples and dunes. Other factors such as turbulence, surface waves, and small irregularities of bed may affect movement of sediment ripples, although they are not primary causes of sediment ripple formation. —*WPA*

Cross-Flow Inlets: A Hydrodynamic Analysis. B. Steenberg & S. Pettersson. Svenska Traforskningsinst., Trakem. o. PappTechn. Medd., No. 209 ('56). Authors give mathematical anal. of flow from uniformly tapped pipe with linearly decreasing crosssectional area, which reveals that there is divergence from constant flow through slit. Authors give results of studies into geome-

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try of pipe which will permit slit of constant width to deliver uniform discharge. Calcns. are given which show that crosssectional area of inlet must decrease less rapidly than is case for pipe with linearly decreasing crosssectional area.—*WPA*

The Boundary Layer Development in Open Channels. J. W. Delleur. Proc. Am. Soc. Civil Engrs., **83**, EM1, Paper No. 1138 ('57). Development of turbulent boundary layer is treated for steady flow in rectangular open channel, and simplified soln. is given for channel of large width. Theory shows interaction of boundary layer and main flow outside layer, with result that layer develops more slowly than for flat plate in infinite fluid under similar flow conditions. For flows close to critical, boundary layer may reach max. and then decrease. For flows less than critical, theory predicts where boundary layer reaches surface. Expts. showed that avg. behavior of boundary layer is in close agreement with theory. Tests revealed that there are secondary motions which affect distr. of velocs. As result, displacement thickness is not constant at any 1 crosssection, but avg. displacement thickness is in close agreement with theory. 2 applications of theory are considered—distance to uniform flow in intake section of open channel, and computation of theoretical discharge coefficient for flows over horizontal broad crested weirs.—*WPA*

Measuring Stream Flow Under Ice Conditions. A. M. Moore. Proc. Am. Soc. Civil Engrs., **83**, HY1, Paper No. 1162 ('57). Effects of ice formation on stage-discharge relation in streams are explained briefly. Occurrence and effect of surface, frazil, and anchor ice are described and shown in graphs, and investigation into "siphon action" caused by surface ice in 1 stream is discussed in detail. Factors affecting accuracy of ice-affected records are considered. —*WPA*

Graphical Determination of Water Surface Profiles. F. F. Escoffier. Proc. Am. Soc. Civil Engrs., **82**, HY6, Paper No. 1114 ('56). Tables of functions for calcg. water surface profiles in open channels of uniform crosssection have been derived by Bakhmeteff, who assumed that conveyance, K, and critical discharge, Q_c, are both exponential functions of depth variable y with same exponent $n/2$ in each case. Author has now developed graphical method which simplifies use of these functions. Advantage of graphical method is that it permits direct soln. for depth of water at end of reach of prescribed length, while in older methods series of trial depths had to be assumed until 1 was found that gave required length.—*WPA*

Flow Measurements in Streams by Addition of Salt. J. Vágás. Hidrol. Kozlony (Hung.), No. 5/6 ('55). After discussing general conditions for measurements of flow in streams by addn. of salt, author gives directions for time during which salt is added, amt. and concn. of salt soln., and length of stretch between place of addn. and place of sampling. Electrical methods of measurement and methods using radioactive materials are described.—*WPA*

The Mean Velocity of Discrete Particles in Turbulent Flow in a Pipe. G. K. Batchelor; A. M. Binnie; & O. M. Phillips. J. Inst. Water Engrs. (Br.), **10**: 476 ('56). Axial component of veloc., averaged over long period of time, of material element of fluid in straight circular pipe is shown theoretically to be equal to discharge veloc. defined as discharge, averaged over a long period, divided by crosssection of pipe. This prediction was confirmed experimentally by injecting in turn into water pipe large number of solid spheres of different sizes, having same density as water. Satisfactory theory is developed to allow for finite size of spheres. Fluctuation in time of travel of particle between 2 fixed stations is examd., and it is shown how few particles may be used to det. dischg. veloc. within calculable limits of accuracy.—*WPA*

Automatic Registering Micro Differential Manometer. O. Falk. Wasserwirtschaft (Ger.), **47**:34 ('56). Detailed description, with diagrams, is given of manometer specially designed for measurement of very low rates of flow in water pipes. Apparatus can also be adapted to higher rates of flow and to measurement of flow of air.—*WPA*

Friction Losses in Water Supply Calculations. H. R. Vallentine. Commonwealth Engr. (Austral.), **44**:77 ('57). Limitations of Manning and Hazen-Williams for-

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mulae in water pipe calcns. are discussed, with particular reference to steady flow in uniform circular pipes flowing full. It is shown that neither of these types of exponential formulae is sufficiently general to cover range of water flow conditions met with in ordinary pipeline calcns. Author concludes that Darcy formula, used with one of conventional graphs relating Darcy coefficient and Reynolds number, is more reliable for all pipe flow calcns. where accurate estimates of loss of head are required. Rough approximations may be obtained with Hazen-Williams formula for cast-iron and steel pipe, and with Manning formula for concrete pipes.—*WPA*

The Dynamic Structure of a Coastal Plain Estuary. D. W. Pritchard. J. Marine Research, **15**:33 ('56). Time mean equations of motion applicable to coastal plain estuary are developed and discussed. Time series observations of temp., salinity, and current veloc. from 2 sections in James R. are used to obtain mean lateral and longitudinal components of relative pressure field, Coriolis force, and field accelerations. Using appropriate boundary conditions, equations of motion are solved for nonadvective or eddy flux of momentum and, from this, depths of pressure surfaces which are level relative to longitudinal and lateral coordinates are found. Variations with depth of longitudinal and lateral components of pressure force are discussed, and relative importance of various terms in equations of motion is evaluated. —*WPA*

A Method for Determining Mean Longitudinal Velocities in a Coastal Plain Estuary. D. W. Pritchard & R. E. Kent. J. Marine Research, **15**:81 ('56). It is shown that longitudinal component of mean veloc. in coastal plain estuary may be computed indirectly. To do this, use is made of lateral and longitudinal components of lateral and longitudinal components of equation of motion, tidal veloc. amplitudes, and deduced relationship between the vertical and lateral eddy stresses. Method is evaluated for station in James R. estuary. Computed veloc. agreed quantitatively with corresponding observed veloc.—*WPA*

Evaluation of Hydrological Investigations. C. Truelsen. Bohrtechn. Brunnenbau, **6**: 337 ('55). Author discusses various processes for calcg. yield of ground water and

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describes process developed by himself which does not require detn. of depth of water-bearing layer and ground water gradient. —*WPA*

Transition Profiles in Nonuniform Channels. F. F. Escoffier. Proc. Am. Soc. Civil Engrs., **82**, HY3, Paper No. 1006 ('56). Transition profile of channel is defined in paper as profile at point where flow changes from tranquil to rapid, or vice versa, without abrupt change. Mathematical interpretations of transition profiles in uniform and nonuniform channels are presented, and graphical method for constructing transition profiles in practical application is outlined.—*WPA*

HYDROLOGY, CONSERVATION & IRRIGATION

Observations on the Use of Hexadecyl Alcohol for Conservation of Water. A. G. Boon & A. L. Downing. J. Inst. Water Engrs. (Br.), **11**:443 ('57). Efficiency of hexadecyl alc. in reducing evapn. depends on intensity of solar radiation, wind veloc., relative humidity, and rate at which films are replaced. It is shown that efficiency of method may be influenced by biol. action. Evapns. from distd. water and natural waters were same. When hexadecyl alc. was added there was difference. Evapn. was less in natural water and noticeably less in contaminated waters.—*CA*

The Use of Hexadecyl Alcohol to Reduce Reservoir Evaporation. F. Grundy. J. Inst. Water Engrs. (Br.), **11**:429 ('57). Hexadecyl alc. was spread on reservoirs either as pellets, as powder, or as soln. in kerosene. Dose of 1 l/acre reduced evapn. by 20%. On 5-acre reservoir dose of 3 l/acre/day gave reduction of 30% of evapn. At Tanganyika, hexadecyl alc. was used for 5 mo. and water supply was used 2 mo. longer than expected. Larger reservoirs might be treated but there are certain problems that require more expts.—*CA*

Chemical Conservation of Water. A. L. Downing & K. V. Melbourne. J. Inst. Water Engrs. (Br.), **11**:438 ('57). Use of hexadecyl alc. to reduce natural evapn. may reduce rate of soln. of O in many, if not in all, waters. Studies of effect on different waters indicate that due regard should be

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given to this effect, particularly in reservoirs that contain high concns. of org. matter, which may decay and cause high O demand.—*CA*

Minimum Water Yield From Small Agricultural Watersheds. L. L. HARROLD. Transactions American Geophysical Union, 38:201 ('57). Methods of analyzing and reporting min. runoff data from small watersheds near Coshocton, Ohio, were illustrated. 12-mo. min. runoff values for 2–100-yr. recurrence intervals, min. runoff values for 1–12-mo. intervals, seasonal occurrence of low runoff values, duration of min. flow, and value of 5-day stream flow minima for Jun., Jul., and Aug. were presented. Basic data were 46 yrs. of pptn. records and 18 yrs. of pptn. and runoff records.—*PHEA*

Closed System Use of Industrial Water. C. J. LEWIS. Proc. 10th Ind. Waste Conf., Purdue Univ., No. 89 ('55). Author stresses importance, in view of increasing shortage of water in some parts of US, of reuse of water by industries.—*WPA*

The Effect of Afforestation Upon the Yield of Water Catchment Areas. F. LAW. J. Brit. Wtr. Wks. Assn., 38:489 ('56). In paper presented at 1956 meeting of British Association for Advancement of Science, expts. are described in which effect of afforestation on water catchment areas was investigated. In spruce plantation in vicinity of Stocks reservoir of Fylde Water Board, measurements were made of rain falling on catchment area, rain reaching ground under trees, rain running down trunks, and of runoff from concrete-enclosed plantation. Figures given in paper show that of 38 in. of rain falling on area, 14 in. was lost by evapn. from tree canopy, and of 24 in. reaching ground only 10.76 in. ran off and was available for supply purposes. Results are discussed in relation to yield from catchment areas and cost of providing supplementary supplies. Data obtained are in harmony with foreign opinion and expts.,

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which conflict with views of British hydrologists.—*WPA*

Investigations of the Hydrology of Small Watersheds in Texas. T. TWICHELL. Proc. Am. Soc. Civil Engrs., 82, SA4, Paper No. 1050 ('56). Author stresses need for continuous inventory of data on water resources as these are fundamental to economy of community. Hydrological investigations, it is suggested, lag behind development and major deficiency is lack of basic data for evaluating natural occurrence of water resources. In Texas, demands for surface water far exceed low flow of every major Texas stream and successful operation of existing water supply projects and proper development of new projects are contingent upon availability of basic hydrological data. Brief anal. is made of hydrological data for Texas watersheds, collected over 2-yr. period. Study does not justify conclusions but illustrates type of data required over longer period to assess true effects of upstream development on runoff.—*WPA*

Some Principles of Agricultural Irrigation. D. R. SISSON. Proc. 10th Ind. Waste Conf., Purdue Univ., No. 89 ('55). Author discusses importance of irrigation in midwest states of US and reviews factors to be considered in designing and operating agricultural irrigation system and in using sprinkler irrigation as means of disposal of waste waters.—*WPA*

Irrigation Requirements Based on Climatic Data. G. H. HARGREAVES. Proc. Am. Soc. Civil Engrs., 82, IR3, Paper No. 1105 ('56). Author discusses limitations of present methods of using climatic data to compute consumptive use and irrigation requirements. Evapn. of water is considered as phys. process, and phys. laws, climatic data, and theoretical considerations are used to derive new equations for detg. consumptive use or evapotranspiration potential for any set of climatic conditions. Formula, based on use of evapotranspiration potentials, is developed for transferring consumptive use data from 1 set of climatic conditions to another. Climatic regions in US are described, and use of consumptive use data in computing irrigation requirements is discussed.—*WPA*

SOFTENING & IRON REMOVAL

Removal of Salts From Water. A. RICHTER. Chem. Zentr. (Ger.), 126:7291 ('55). For removal of salts, water is treated with hydrogen-ion and hydroxyl-ion exchange materials. Filter for adjusting pH value follows exchange filter system. First filter may contain cation-exchange material made from resorcinol or methyl resorcinol sulfonic acid and formaldehyde, regenerated with hydrochloric acid, and second, anion-exchange material from *m*-phenylene diamine, polyethylene diamine, and formaldehyde, regenerated with sodium hydroxide free from calcium and magnesium. Third filter may contain anion-exchange resin of same constitution as that of second filter, which has been freed from soluble substances, such as salts, acids, and lyes, by washing with hardness-free water.—*WPA*

Dimensions and Arrangement of Plants for Complete Removal of Salts. H. LIST. Vom Wasser (Ger.), 23:265 ('56). Author gives illustrated account of various possible arrangements of cation- and anion-exchange filters in plant for complete demineralization of water, and discusses possible methods of economy in construction and operation costs and purpose and advantages of insertion of mixed filter as final step in process.—*WPA*

Pure Water by Ion Exchange. Chem. Trade J. (Br.), 140:864 ('57). Portable app. for demineralization of water, Mark V Portable "Deminrolit," is described. It consists of 2 columns of ion-exchange material, first containing cation-exchange resin Zeo-Karb and second, anion-exchange resin De-Acidite. App. has output of 6 gph.—*WPA*

The Demineralization of Water in Mixed Beds. J. PIERREY. Tech. Eau (Belg.), No. 103, p. 39 ('55). Operation of mixed-bed ion-exchange units for demineralization of water is described briefly and advantages of mixed-bed units over 2-stage units are indicated.—*WPA*

Demineralized Water. II. The Chemical and Bacteriological Quality of Demineralized Water Produced in a Central Location and Distributed Through Pipes. P. RASMUSSEN. Arch. Pharm. Chemi (Den.), 64:256 ('57). Description of cation-anion-cation exchange, 3-tower, centrally lo-

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cated set-up. Daily use of demineralized water was about 2,000 l. Chem. qual. corresponded to about 1,000,000 ohm-cm. Bacteriologically it was satisfactory, but pyrogens were occasionally encountered.—*CA*

Operating Hydrogen Decalcification Plants. J. SCHMIDT. Mitt. Ver. Grosskesselbesitzer, **47**:117 ('57). H-exchange method is widely in use for removing CO_2 or salts. Plants need intensive maintenance. Principle of the process is to replace Ca, Mg, or Na ions by H ions. In operating plants Ca as well as Cl^- and SO_4^{--} content should be checked repeatedly.—*CA*

Starting Up and Operating a Water Desalting System in the Mosener Water System. L. A. CHERNOVA & M. M. GUSHCHINA. Energetik (USSR), **5**:11 ('57). Discussion of design and operating details of plant for conditioning feed water for boiler operating under 185 atm. In flow order are primary cation and anion exchangers, decarbonator, and secondary cation and anion exchangers. Raw water for boiler feed is converted from (mg/l): suspended matter 4.5–8.4 to 0; salt content 140–257 to 0.02–0.05; total hardness 1.7–7.9 to 2.4; Cl^- 2.2–5.3 to 0; SO_4^{--} 24–35 to trace.—*CA*

WELLS & GROUND WATER

The Law of Filtration in Ground-Water Movement. O. CZEPA. Acta Hydrophys., 3:181 ('56). Author discusses application of laws of filtration to ground water movement and relevant methods of calcn.—*WPA*

Ground Water as Part of the General Water Supply. A. WECHMANN. Gas- u. Wasserfach. (Ger.), **97**:601 ('56). In general discussion of importance of ground water in general water supply, author first deals with the conception of ground water and methods of detg. whether water is true ground water or is affected by surface waters. Subjects then considered include various factors, natural and artificial, affecting flow and condition of ground waters, amt. of ground water available and amt. used for supplies for different purposes in Germany, calcn. of future demand for water for various purposes, plans for future, and research required into conditions of ground water, both in quan. and qual. Difficulties in ground water investigations as compared with investigations into surface water are discussed.—*WPA*

Water Supplies From Wells. H. O. WILLIAMS. Off. Bul. N.D. Wat. Wks. Conf., **24**:2 ('57). This illustrated article compares wells in rock and in sand formations, and gives an account of operation of latter type of well. Cause of drawdown and importance of proper construction to avoid excessive losses in head are explained. Methods of well construction in different types of sand and gravel beds and influence of well diam. and screen length on yield are discussed. Details are given of control and treatment of corrosion and incrustation of well screens.—*WPA*

The 1954 Drought and Its Effect on Ground Water. G. A. MUILENBURG. Proc. Am. Soc. Civil Engrs., **82**: SA3, Paper No. 1016 ('56). Drought conditions prevailed in Mo. and neighboring areas from 1952 to 1954, and reached max. in summer of 1954. Effect of drought on ground water in Mo. is discussed. Ground water in shallow zone was below normal but, in general, deep water-bearing strata were not affected.—*WPA*

Directions for Supplementing Ground Water and the Production of Artificial Ground Water. C. TRUELSEN. Bohrtechn.-Brunnenbau, **7**:181 ('56). Various methods for supplementing flow of ground water are described and hydrology of process is discussed.—*WPA*

Importance of Ground Water in Our National Economy. A. G. FIEDLER. J. Sanit. Eng. Div., Proc. Am. Soc. Civil Engrs., **83**: 1271 ('57). Use of water derived from underground sources by means of wells has been increasing greatly in recent yrs. Though ground water meets only about 15% of nation's total requirements for water, it is an important resource upon which large segment of our pop. depends. There is no national ground water problem as such, but there are numerous widely scattered problem areas, each differing more or less markedly from its neighors because of distinctive geologic and hydrologic environment and other local factors. Typical ground water problems in 6 areas are reviewed and it is concluded that soln. of each must rest on ade-

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quate knowledge of character and capac. of ground water reservoirs involved and on comprehensive planning for best use of available ground water supply. To solve many of major problems will be complex and expensive, and practically all will require considerable public education and support if adequate soln. is to be achieved.—*PHEA*

Protective Zones for Horizontal Filter Wells. G. KELLER. Gas- u. Wasserfach. (Ger.), **98**:82 ('57). Author deals first with special difficulties of detg. and controlling protective zones for wells with horizontal filters, with special reference to districts with medium and small farms such as are found in northwest Germany. In detg. protective zones, not only well itself but filter pipes and especially pipe heads must be taken into account. Extent of 3 protective zones should be calcd. for each case according to regulations of Deutsche Verein von Gas- und Wasserfachmannern. With horizontal filter wells, intake funnel formation of ground water surface is replaced by basin-like depression.—*WPA*

"Safe Yield" in Ground Water Development—Reality or Illusion? R. G. KAZMANN. Proc. Am. Soc. Civil Engrs., **82**: IR3, Paper No. 1103 ('56). Author exams. critically definitions of "safe yield" of aquifers, and concludes that they are unsatisfactory. It follows that much of existing legislation concerning ground water will prove to be administratively unworkable and inequitable. Future legislation should be based on appropriation doctrine, and consideration should be given to possibility of development of generally applicable method of artificial recharge using raw surface water that is either untreated or has received an abs. min. of treatment.—*WPA*

Ground Water Investigations of a Special Type in the Cologne Area. G. SCHROEDER. Gas- u. Wasserfach. (Ger.), **97**:854 ('56). Investigation is described of effects of open mining in Cologne area on condition of ground water. Geological and hydrological conditions and methods of calcg. amts. and direction of ground water flow are discussed. In general, calcns. show that withdrawal of ground water from mining area will result in reversal of ground water stream so that between Rhine and mining area flow will consist entirely of filtered Rhine water unmixed with true ground water.—*WPA*

(Continued from page 46 P&R)

Sikkim on the rocks isn't a villain's command to his dogs but a drink that may one day be the bartender's best seller. It may—as a matter of fact, it should—be a while yet, as the distillery that will produce Sikkim is yet to be built. It will be built, of course, in Sikkim, a tiny Himalayan kingdom which may have escaped your notice, as part of the 7-year plan of Premier Rustomjee Dewan, whose advisers have informed him that Sikkim's water so closely resembles the water of Scotland that it would not be difficult to distill good quality Scotch (Sikkim) there. With the hope that Sikkim will become an important Sikkim export, the government has given top priority to the construction of the distillery. As our only knowledge of the Himalayas comes from James Hilton's *Lost Horizon,* we're just a little bit worried about how Sikkim is going to be aged. Ah well, if it isn't, *Lost Weekend!*

The rocks, by the way, under Scotch—and, certainly, similarly, Sikkim—should not, according to a note in the December 1957 issue of *Cosmopolitan,* be hard, lest they damage the drink. To be "not hard," of course, is not to be "not frozen," but to be frozen from water that is not hard. A full explanation, we understand, is obtainable during trans-Atlantic flights from BOAC stewardesses. *Sic 'em!*

John A. Andrea, Chapel Hill, N.C., water and sewer department chief engineer, has been named to the newly created post of chief city engineer in charge of all street, utility, and line work. Mr. Andrea, who has been working for the city since 1947, will continue with his water and sewer duties.

(Continued on page 82 P&R)

(*Continued from page 80 P&R*)

'Death by drowning' is rarely found in a water utility accident report, but it happened (not in line of duty) last Dec. 16 at Cumming, Ga. Earl Hamby had stopped in at the city water plant that night to make a phone call. Disturbed by the noise of the pumps, he carried the telephone to a nearby doorway and stepped through it. The pump intake basin was on the other side.

Frank B. Sarles' listing in the 1957 AWWA Directory of Consultants failed to include the notation that he is a registered professional engineer in California. Mr. Sarles is a consulting civil engineer, with offices at 425 South E Street, Santa Rosa, Calif.

The first award of the William M. Cobleigh Scholarship has been presented to Floyd K. Haugen, of Belt, Mont., a senior engineering student at Montana State College, Bozeman. The Montana Section of AWWA and the Montana Sewage & Industrial Wastes Assn. were instrumental in establishing the scholarship fund as a memorial to Dean Cobleigh, one of the founders of both organizations, who died in 1951.

Francis L. Winslow has been elected president of the Plainfield-Union Water Co., Plainfield, N.J. Mr. Winslow, secretary-treasurer of the company for 30 years, succeeds William T. Speer, who resigned last May.

(*Continued on page 84 P&R*)

(*Continued from page 82 P&R*)

A dry chemical feeder with a self-cleaning feed mechanism has been introduced by Wallace & Tiernan Inc., Belleville, N.J. Constructed of heavy-gage steel, the A-690 feeder has a hopper capacity of 3.25 cu ft and a feed rate up to 2.85 cu ft/hr. The self-cleaning feed screw discharges the

chemical alternately from each end of a feed tube. While one end of the screw is discharging, the other end is cleaned by being threaded through the material. A removable plastic window on the front of the volumetric feeder (see photo) permits observation of the feed screw in operation.

(*Continued on page 86 P&R*)

(*Continued from page 84 P&R*)

Harvey F. Ludwig and Joseph L. Feeney have formed the consulting engineer firm of Ludwig Engineering, with offices at 490 E. Walnut St., Pasadena, Calif.

Legal notes: *Fluoridation.* The New York State Supreme Court has dismissed an application of a number of customers of the New Rochelle Water Co. to set aside a State Public Service Commission rate order. The customers had sought to reverse the commission's decision to allow fluoridation costs to be included in the rate base.

Taxation. The New Jersey Supreme Court ruled that a municipality cannot tax water companies for the value of water flowing through mains. The court upheld decisions by the State Div. of Tax Appeals and the Hudson County Tax Board which canceled tax bills of $200,000 a year from 1952 to 1955 against the Hackensack Water Co. The court ruled that water in the mains is not the personal property of the utility distributing it.

An achievement in achievement is being undertaken this year by the Indianapolis Water Co. as sponsor of a group of tyro tycoons who have established a corporation to manufacture and sell toy chests under the name of Tru-Kut Products. Set up under the guidance of Junior Achievement—a national nonprofit organization established in 1919 to teach youth the free-enterprise system on a practical

(*Continued on page 88 P&R*)

Drawing showing unbalanced pressures acting on impeller and shaft in single volute pump whenever it is operating at below peak efficiency.

Drawing showing how radial forces are equalized in Wheeler-Economy Dual Volute Centrifugal Pumps. Note that the inlet for each volute is 180° from the other.

How Wheeler-Economy

Dual Volute Design

prolongs pump life by equalizing radial forces acting on pump impeller and shaft

Operating high-head, high-capacity centrifugal pumps at less than peak efficiency—even intermittently—can cause considerable trouble. Forces of five to ten times the weight of the rotating parts are set up, often with the result that the pump shaft breaks, the casing rings wear prematurely and the stuffing box leaks.

Wheeler-Economy Dual Volute design solves these problems by forcing the liquid to accelerate and decelerate at a uniform rate regardless of load. As you can see from the sketch, liquid leaving the impeller at the "nine o'clock" position enters one volute; liquid leaving at "three o'clock" enters the other. In this way, radial pressures at each point along the impeller periphery are balanced by equal and diametrically opposite pressures—eliminating eccentric wear of stationary parts and pump shaft fatigue failure.

Fill out and mail the coupon below for complete information on Wheeler-Economy Dual Volute Centrifugal Pumps. Or see your representative.

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Centrifugal, Axial and Mixed Flow Pumps • Steam Condensers • Vacuum Equipment • Marine Auxiliary Machinery • Nuclear Products

(Continued from page 86 P&R)

basis—Tru-Kut Products is one of 25 such corporations that were started in Indianapolis last October and will be dissolved next May. Each of them manufactures and sells some product. The Junior Achievers in each group constitute their own board of directors, elect themselves to management positions, and work in production and sales departments, paying themselves a salary for duties performed. As Junior Achievement is operated on a nonprofit basis, the corporations are exempt from federal income tax, in lieu of which they pay 30 per cent of their profits to the national organization.

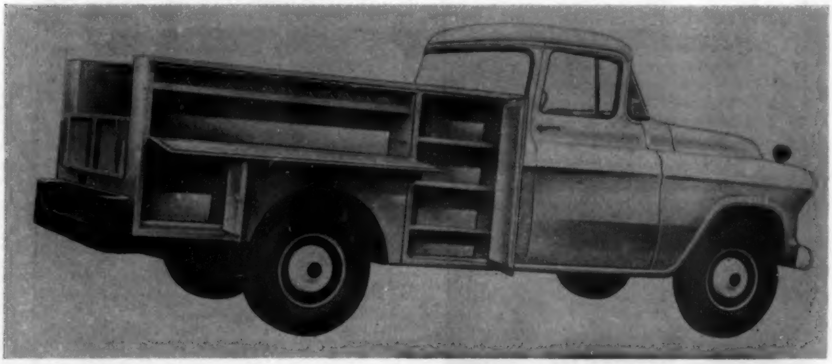
As a sponsor, the water utility provides three advisers—on sales, business, and production—to Tru-Kut and contributes minor incidental expenses. The funds for setting up the project as a whole, totaling $28,000, were raised by contributions from firms which do not actively participate in the operation. There, too, the Indianapolis Water Co. helped, by contributing the services of its treasurer as a fund raiser. And as first customer, President Tom Moses is now known as the man with the Tru-Kut chest.

Not on a nonprofit basis, the utility takes its kut in community relations!

Arthur W. Radford, former chairman of the US Joint Chiefs of Staff, has been elected a director of Worthington Corp., Harrison, N.J. Admiral Radford recently retired from the Navy after 45 years of service.

W. E. MacDonald has retired as commissioner of water works at Ottawa, Ont., after 47 years of service to the city. He plans to engage in consulting work.

Dudley R. Shepard has retired as town engineer and superintendent of public works for Nutley, N.J. Succeeding him as superintendent is Harry Paxton, senior engineer in the department. Mr. Shepard expects to engage in consulting work.



Steel truck bodies featuring variable shelving arrangements and capable of being mounted on any standard chassis have been designed especially for water and sewerage system maintenance by Reading Body Works, Inc., Reading, Pa.

HOW TO SELECT WATER-CONDITIONING EQUIPMENT (NO. 7 OF A SERIES)

Water Impurities and Methods of Treatment*

HARDNESS *(Scale)* Calcium or magnesium salts such as bicarbonates, chlorides and sulfates	**Ion Exchange** (zeolite softeners). Generally used on clean, clear water to reduce hardness to a very low figure. Also avoids repumping. **Precipitator** (cold lime-soda softening). Used where turbidity, color or iron or manganese are also to be removed. Sometimes used for softening only—gives lower operating cost than ion exchange on high-carbonate-hardness waters. Reduces hardness to a few grains per gallon. Can be followed by zeolite softeners. **Hot-process Softener** (lime-and-soda process) usually followed by anthracite filter. Used for boiler feedwater. Also removes silica. Reduces hardness to a few grains per gallon. If followed by a zeolite softener (optional), no soda ash is used.
TURBIDITY Suspended dirt, sand, silt or other solids	**Sand Filter** (gravity or pressure type) alone . . . for water with moderately low turbidity content. **Precipitator** will reduce bulk of turbidity; generally followed by sand or anthracite filter for practically complete removal.
COLOR Dissolved or finely divided organic matter	A **Precipitator** with alum coagulant is generally used. Powdered activated carbon is sometimes added for further reduction of color. **Chlorination** may be used to kill living organic matter.
BAD TASTE OR ODOR Hydrogen sulfide (H_2S), cause of "rotten egg" odor	**Aeration** with several hours detention. **Adding chlorine:** Expensive except for removing small residuals. **Forced-draft degasifier:** Used if there are 10 or more ppm of H_2S or if CO_2 is also to be removed. **Carbon Purifier** (Carbo Dur®) or **Manganese Zeolite Filter:** For small volumes of water with 2 ppm or less H_2S.
Algae, organic matter	Same treatment as for COLOR. A **Carbon Purifier** may be used for small volumes of water . . . preceded by a sand filter if turbidity or debris is to be removed.
Excess chlorine	A **Carbon Purifier** provides simple, automatic operation. Also removes other causes of tastes and odors. **Sodium Sulfite** treatment reduces chlorine at lower equipment cost.
CORROSIVENESS	Neutralize with caustic soda, soda ash or lime.

Section Meetings

Virginia Section: The 24th annual meeting of the Virginia Section was held at Hotel Roanoke, Roanoke, Nov. 6–8, 1957, with a record attendance of 241. The highest previous attendance was 237, in 1952. The meeting opened on a note of sadness as the result of the death of Linn Enslow at his farm in Pulaski County, Va., on Sunday, Nov. 3. As was his custom, Linn usually visited his farm a few days in advance of the Virginia Section meeting, and his attendance was always anticipated. Many of the members did not know of his death until the meeting started. The Section had been saddened earlier in the year by the death of Herman W. Snidow, regional engineer with the State Dept. of Health, who was a charter member of the Section and its first secretary-treasurer.

With the excellent facilities of Hotel Roanoke and with a well varied program, the technical sessions were well attended. In line with the current emphasis on safety, the program included a panel discussion on this subject headed by X. D. Murden, chairman of the Section's Accident Prevention Committee. An operators' certification program outlined by C. G. Haney, superintendent of filtration, Charlottesville, evoked considerable discussion. The technical sessions closed on Friday with a humorous and philosophical talk by Walter A. Flick, of Washington and Lee University, stressing the advantage of pleasant relations with people, both in business and across the back fence. [A complete list of technical papers presented appeared on p. 1615 of the December 1957 issue.]

A field trip to the Norfolk & Western Railway shops was enjoyed by a large number of the men. This is one of the few railroads, if not the only one, where large steam locomotives are both constructed and maintained. The guided tour was revealing to all. To a great many persons, a locomotive will always be more fascinating and romantic than jet planes or ballistic missiles.

As usual, a large number of ladies were present. Over 50 were entertained at a luncheon at Bennett Springs on the

(*Continued on page 94 P&R*)

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ALTITUDE VALVE

Controls elevation of water in tanks, basins and reservoirs

1. Single Acting
2. Double Acting

Maintains safe operating pressures for conduits, distribution and pump discharge

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REDUCING VALVE

Maintains desired discharge pressure regardless of change in rate of flow

Regulates pressure in gravity and pump systems; between reservoirs and zones of different pressures, etc.

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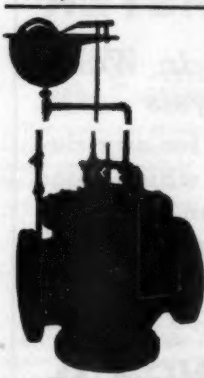
Combination automatic control both directions through the valve.



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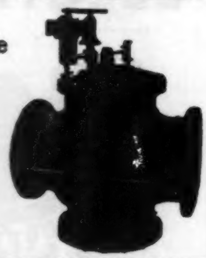
Electric remote control—solenoid or motor can be furnished



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Section Meetings

(*Continued from page 92 P&R*)

first day of the meeting, and there was a card party at the hotel the following day. As is customary, each lady received a corsage with the compliments of the Section prior to the annual banquet.

During the banquet, Old Dominion Award certificates were presented to: J. H. Gardner, president and treasurer, Bingham & Taylor Corp., Culpeper; E. C. Meredith, director, Div. of Engineering, State Dept. of Health, Richmond; J. M. Pharr, assistant general manager, Newport News Water Works Commission, Denbigh; and Capt. Paul E. Seufer, US Navy, Washington, D.C. The Old Dominion Award is for continuous membership in AWWA for 20 years.

In the first year of the safety program, it was gratifying that Safety Awards were made to six Virginia water departments. Portsmouth received the Award of Honor, and the Award of Merit went to Winchester, Staunton, Martinsville, Fries, and Franklin.

The Virginia Section was happy to have a Fuller Award nominee this year, in the person of Harry N. Lowe Jr., chief, Sanitary Engineering Branch, US Army Engineer Research & Development Labs., Fort Belvoir. The recognition was never more deserved.

A dance which followed the banquet was enlivened considerably by the presence of Bill Orchard and Fred Merryfield, with the former at the piano and the latter at the microphone leading group singing. This pair has worked together before, which no doubt accounts for the professional nature of the performance. The fact that the Manufacturers' Club Room had set up shop in the ballroom for the duration of the dance may have helped.

J. P. Kavanagh
Secretary-Treasurer

MEMBERSHIP METER
TOTAL
11|8|3|1
FEB | 1 | 1958
AMERICAN WATER WORKS ASSOCIATION

NEW MEMBERS

Applications received Jan. 1–31, 1958

Allen, William L., Jr., Pres., Western Filter Co., 4545 E. 60th Ave., Denver 16, Colo. (Jan. '58) *P*

American Machine & Foundry Chem. Research & Dev. Lab., W. K. W. Chen, 689 Hope St., Springdale, Conn. (Corp. M. Jan. '58) *P*

Ashton, Dan P., Tech. Supervisor, Crown Zellerbach Corp., St. Helens, Ore. (Jan. '58) *MP*

Bailey, James W., Plant Operator, Clarksville Light & Water Co., Clarksville, Ark. (Jan. '58) *P*

Barrett, Raymond, Sr. Exec. Engr., Public Works Dept. Federal Headquarters, Maxwell Rd., Kuala Lumpur, Malaya (Jan. '58) *PD*

Bell, Howard Otis, Water Supt., Yellville, Ark. (Jan. '56)

Bookwalter, W. F., Vice-Pres., Boyd E. Phelps, Inc., 1000 Washington St., Michigan City, Ind. (Oct. '56)

Boydston, James R., Project Engr., Stevens & Thompson, 2234 S.W. 5th St., Portland 7, Ore. (Jan. '58) *P*

Brewer, Richard N., Gen. Mgr., Boyd E. Phelps, Inc., 1000 Washington St., Michigan City, Ind. (Jan. '54)

Bristol Co., The, G. P. Lonergan, Sales Promotion Mgr., Waterbury 20, Conn. (Assoc. M. Jan. '58)

Bucher, Robert W., Service Director, City Bldg., High St., Wadsworth, Ohio (Jan. '58)

Casey, Thomas J., San. Engr., Ford, Bacon & Davis, 39 Broadway, New York, N.Y. (Jan. '58) *RPD*

Chas, Andrew C., Draftsman, Bell Telephone Labs., Murray Hill, N.Y. (Jan. '58) *PD*

Chen, W. K. W.; *see* American Machine & Foundry Chem. Research & Dev. Lab.

Crowley, Frank W., Acting Sr. Exec. Engr., Office of ADPW Water Supplies, PWD Federal Headquarters, Maxwell Rd., Kuala Lumpur, Malaya (Jul. '54)

Dahlgren, William J., Chief Engr., Tenn. Inspection Bureau, 1000 Stahlman Bldg., Nashville 3, Tenn. (Jan. '58) *D*

Davis, City of, Frank Fargo, City Administrator, City Hall, Davis, Calif. (Munic. Sv. Sub. Jan. '58) *MD*

Diefendorff, Adelbert, Dean, School of Eng., College of the Pacific, Stockton 4, Calif. (Jan. '58) *MP*

Dixon, R. M., Chairman, State Bd. of Water Engrs., 1410 Lavaca St., Austin 1, Tex. (Apr. '38)

Eldridge, L. C., City Water Bd., Box 2449, San Antonio 6, Tex. (Jul. '42)

Fargo, Frank; *see* City of Davis (Calif.)

Fuller, Emmett J., Supt. of Utilities, Water & Elec. Dept., Ellis, Kan. (Jan. '58) *MD*

Fulmer, E. C., Dist. Service Engr., Electric Chem. Co., 8011 Franklin Blvd., Cleveland, Ohio (Jan. '58) *RPD*

Gardner, Marvin B., Jr., Chief Engr., Morland Assocs., 63 South Ave., Garwood, N.J. (Jan. '58) *RPD*

Gengo, Joseph T., Asst. Supt., Water & Light Dept., Salamanca, N.Y. (Jan. '58) *MPD*

Gove, Robert H., Civ. Engr., 149 S. Edwards St., Kalamazoo, Mich. (Jan. '58) *MRPD*

Graham, H.; *see* Victoria (B.C.) Public Utilities Com.

Green, Norman S., Editor, Export Publisher's, 134 E. 59th St., New York 22, N.Y. (Jan. '54)

Gumbel, Walter C., Conservationist, Monongahela Power Co., Box 1392, Fairmont, W.Va. (Jan. '58) *RPD*

Hancock, Robert M., Sales Repr., Western Utilities Supply, Box 3524, Seattle, Wash. (Jan. '58) *D*

Haney, Ralph M., Office Mgr., Sante Fe Irrigation Dist., Box 408, Rancho Santa Fe, Calif. (Jan. '58) *MD*

Harris, Sam., Distr. Supt., Clarksville Light & Water Co., Clarksville, Ark. (Jan. '58) *D*

Henderson, William L.; *see* Orange County (Calif.) San. Dists.

Hughes, Bernard F., Jr., Product Mgr., Fabricated Plate Products, Alco Products, Inc., Dunkirk, N.Y. (Jan. '58)

Johnson, Lewis H., Jr., Sales Engr., B-I-F Texas, Inc., North Central Expressway, Dallas, Tex. (Oct. '54)

Lambirth, Donald L., Sales Repr., Western Utilities Supply Co., Box 3524, Seattle 24, Wash. (Jan. '58)

Lonergan, G. P., *see* The Bristol Co.

Miller, John S., Acting Div. Sales Mgr., American Pipe & Const. Co., Box 630, Hayward, Calif. (Jan. '58) *D*

Morton, William, Cons. Engr., 920 Miller St., Seattle 2, Wash. (Jan. '58)

Nauta, Albert J., Asst. Sales Mgr., Hungerford & Terry, Inc., Clayton, N.J. (Jan. '58) *P*

Norris, John R., Chief Operator, Filter Plant, Clarksville Light & Water Co., Clarksville, Ark. (Jan. '58) *P*

O'Brien, Francis J., Supt., Water Dept., 11 City Hall, Burlington, Vt. (Jan. '58) *MPD*

Oppelt, Victor H., Shop Supt., Meter Div., Water Dept., 531 E. 29th St., San Bernardino, Calif. (Jan. '58)

Orange County San. Dists., William L. Henderson, Chief Chemist, Rm. 239, 1104 W. 8th St., Santa Ana, Calif. (Munic. Sv. Sub. Jan. '58)

Palmer, F. F., City Engr., Box 486, Forsyth, Mont. (Jul. '34)

Paquette, Wilfrid S., Sales Engr., Wallace & Tiernan, Inc., Newark, N.J. (Jan. '58) *P*

Potter, Albert M., Partner, Servisoft Soft Water Service, 217—4th Ave., S., Clinton, Iowa (Jan. '58) *P*

Price, William P., Jr., Gen. Mgr., United Water Conservation Dist., 806 Railroad Ave., Santa Paula, Calif. (Jan. '58) *RD*

Raines, Harold, Attorney East Bay Munic. Utilities, Dist., 2130 Adeline St., Oakland 23, Calif. (Jan. '58) *R*

Roark, Chris C., Foreman, Neches Butane Products Co., Port Neches, Tex. (Apr. '47)

Slee, Angus E., Transite Pipe Mgr., Johns-Manville Sales Corp., 301 Continental Oil Bldg., Denver 2, Colo. (Jan. '58) *D*

Smith, Albert R., Contracting Engr., Pittsburgh Des Moines Steel Co., 1015 Tuttle St., Des Moines, Iowa (Jan. '56)

Smith, Cicero, Supt. of Public Works, Forest Grove, Ore. (Jan. '54)

Smith, Sherman A., Cons. Engr., Sherman Smith & Assocs., 921 Summer St., Burlington, Iowa (Jan. '58) *MRPD*

Soderberg, Arthur L., 83 Farley Ave., Fanwood, N.J. (Oct. 51)

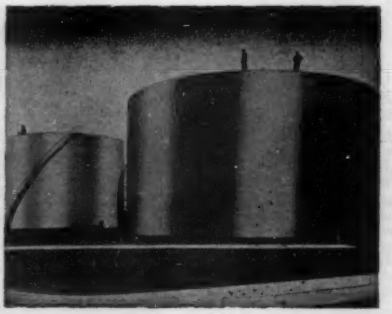
Sommer, Adam, Application Engr., Alco Products, Inc., 25 Greco Lane, Dunkirk, N.Y. (Jul. '55)

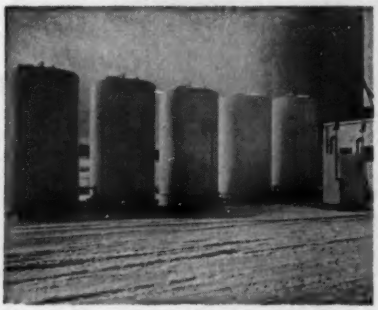
Taggart, Eugene J., Sales Engr., Graver Water Conditioning Co. 216 W. 14th St., New York 11, N.Y. (Jan. '58) *RP*

Thompson, Inman Holt, Supt., Parkin Water Co., Parkin, Ark. (Jan. '56)

Victoria Public Utilities Com., H. Graham, Chief Engr., 503 Central Bldg., Victoria, B.C. (Corp. M. Jan. '58)

Wagner, Harold F., Supt. of Public Works, 203 Chestnut St., Dowagiac, Mich. (Jan. '58) *M*





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Sparling Meter Co.
Wallace & Tiernan Inc.

Gages, Loss of Head, Pressure of Vacuum, Rate of Flow, Sand Expansion:
Builders-Providence, Inc. (Div., B-I-F Industries, Inc.)
Burgess-Manning Co., Penn Instruments Div.
Foxboro Co.
Infilco Inc.
Minneapolis-Honeywell Regulator Co.
Jos. G. Pollard Co., Inc.
Simplex Valve & Meter Co.
Wallace & Tiernan Inc.

Gasholders:
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
Graver Tank & Mfg. Co.
Hammond Iron Works
Pittsburgh-Des Moines Steel Co.

Gaskets, Rubber Packing:
James B. Clow & Sons
Johns-Manville Corp.

Gates, Shear and Sluice:
Armco Drainage & Metal Products, Inc.
Chapman Valve Mfg. Co.
James B. Clow & Sons
Mueller Co.
R. D. Wood Co.

Gears, Speed Reducing:
DeLaval Steam Turbine Co.
Worthington Corp.

Glass Standards—Colorimetric Analysis Equipment:
Klett Mfg. Co.
Wallace & Tiernan Inc.

Goosenecks (with or without Corporation Stops):
James B. Clow & Sons
Hays Mfg. Co.
Mueller Co.

Hydrants:
James B. Clow & Sons
Darling Valve & Mfg. Co.
M. Greenberg's Sons
Kennedy Valve Mfg. Co.
Ludlow Valve Mfg. Co., Inc.
M & H Valve & Fittings Co.
Mueller Co.
A. P. Smith Mfg. Co.
Rensselaer Valve Co.
R. D. Wood Co.

Hydrogen Ion Equipment:
W. A. Taylor & Co.
Wallace & Tiernan Inc.

Hypochlorite; see Calcium Hypochlorite; Sodium Hypochlorite

Ion Exchange Materials:
Allis-Chalmers Mfg. Co.
Cochrane Corp.
General Filter Co.
Graver Water Conditioning Co.
Hungerford & Terry, Inc.
Infilco Inc.
Permutit Co.
Roberts Filter Mfg. Co.

Iron, Pig:
Woodward Iron Co.

Iron Removal Plants:
American Well Works
Chain Belt Co.
Cochrane Corp.
General Filter Co.
Graver Water Conditioning Co.
Hungerford & Terry, Inc.
Infilco Inc.
Permutit Co.
Roberts Filter Mfg. Co.
Walker Process Equipment, Inc.

Jointing Materials:
Johns-Manville Corp.
Keasbey & Mattison Co.
Leadite Co., Inc.

Joints, Mechanical, Pipe:
American Cast Iron Pipe Co.
Cast Iron Pipe Research Assn.
James B. Clow & Sons
Dresser Mfg. Div.
Trinity Valley Iron & Steel Co.
United States Pipe & Foundry Co.
R. D. Wood Co.

Leak Detectors:
Jos. G. Pollard Co., Inc.

Lime Slakers and Feeders:
Dorr-Oliver Inc.
General Filter Co.
Infilco Inc.
Omega Machine Co. (Div., B-I-F Industries, Inc.)
Permutit Co.
Wallace & Tiernan Inc.

Magnetic Dipping Needles:
W. S. Darley & Co.

Meter Boxes:
Ford Meter Box Co.
Pittsburgh Equitable Meter Div.

Meter Couplings and Yokes:
Badger Meter Mfg. Co.
Dresser Mfg. Div.

Ford Meter Box Co.
Hays Mfg. Co.
Hersey Mfg. Co.
Mueller Co.
Neptune Meter Co.
Pittsburgh Equitable Meter Div.
Worthington-Gamon Meter Co.

Meter Reading and Record Books:
Badger Meter Mfg. Co.

Meter Testers:
Badger Meter Mfg. Co.
Ford Meter Box Co.
Hersey Mfg. Co.
Neptune Meter Co.
Pittsburgh Equitable Meter Div.

Meters, Domestic:
Badger Meter Mfg. Co.
Buffalo Meter Co.
Hersey Mfg. Co.
Neptune Meter Co.
Pittsburgh Equitable Meter Div.
Well Machinery & Supply Co.
Worthington-Gamon Meter Co.

Meters, Filtration Plant, Pumping Station, Transmission Line:
Builders-Providence, Inc. (Div., B-I-F Industries, Inc.)
Burgess-Manning Co., Penn Instruments Div.
Foster Eng. Co.
Infilco Inc.
Minneapolis-Honeywell Regulator Co.
Simplex Valve & Meter Co.
Sparling Meter Co.

Meters, Industrial, Commercial:
Badger Meter Mfg. Co.
Buffalo Meter Co.
Builders-Providence, Inc. (Div., B-I-F Industries, Inc.)
Burgess-Manning Co., Penn Instruments Div.
Hersey Mfg. Co.
Neptune Meter Co.
Pittsburgh Equitable Meter Div.
Simplex Valve & Meter Co.
Sparling Meter Co.
Well Machinery & Supply Co.
Worthington-Gamon Meter Co.

Meter Repair Parts
Meter Specialty Co.

Mixing Equipment:
Chain Belt Co.
General Filter Co.
Infilco Inc.
F. B. Leopold Co.

Paints:
Barrett Div.
Inertol Co., Inc.
Koppers Co., Inc.

Pipe, Asbestos-Cement:
Johns-Manville Corp.
Keasbey & Mattison Co.

Pipe, Brass:
American Brass Co.

Pipe, Cast Iron (and Fittings):
Alabama Pipe Co.
American Cast Iron Pipe Co.
Cast Iron Pipe Research Assn.
James B. Clow & Sons
Trinity Valley Iron & Steel Co.
United States Pipe & Foundry Co.
R. D. Wood Co.

Pipe, Cement Lined:
American Cast Iron Pipe Co.
Cast Iron Pipe Research Assn.
James B. Clow & Sons
United States Pipe & Foundry Co.
R. D. Wood Co.

Pipe, Concrete:
American Concrete Pressure Pipe Assn.
American Pipe & Construction Co.
Lock Joint Pipe Co.

Pipe, Copper:
American Brass Co.

Pipe, Steel:
Alco Products, Inc.
Armco Drainage & Metal Products, Inc.
Bethlehem Steel Co.

Pipe Cleaning Services:
National Water Main Cleaning Co.

Pipe Cleaning Tools and Equipment:
Flexible Inc.

Pipe Coatings and Linings:
American Cast Iron Pipe Co.
Barrett Div.
Cast Iron Pipe Research Assn.
Centriline Corp.
Inertol Co., Inc.
Koppers Co., Inc.
Reilly Tar & Chemical Corp.

Pipe Cutters:
James B. Clow & Sons
Ellis & Ford Mfg. Co.
Jos. G. Pollard Co., Inc.
A. P. Smith Mfg. Co.

Pipe Jointing Materials; see Jointing Materials

Pipe Locators:
W. S. Darley & Co.
Jos. G. Pollard Co., Inc.

Plugs, Removable:
James B. Clow & Sons
Jos. G. Pollard Co., Inc.
A. P. Smith Mfg. Co.

Potassium Permanganate:
Carus Chemical Co.

Pressure Regulators:
Allis-Chalmers Mfg. Co.
Foster Eng. Co.
Golden-Anderson Valve Specialty Co.
Mueller Co.
Ross Valve Mfg. Co.

Pumps, Boiler Feed:
Allis-Chalmers Mfg. Co.
DeLaval Steam Turbine Co.
Layne & Bowler Pump Co.
Worthington Corp.

Pumps, Centrifugal:
Allis-Chalmers Mfg. Co.
American Well Works
DeLaval Steam Turbine Co.
C. H. Wheeler Mfg. Co.
Worthington Corp.

Pumps, Chemical Feed:
Infilco Inc.
Proportioneers, Inc. (Div., B-I-F Industries, Inc.)
Wallace & Tiernan Inc.

Pumps, Deep Well:
American Well Works
Layne & Bowler, Inc.
Layne & Bowler Pump Co.
Worthington Corp.

Pumps, Diaphragm:
Dorr-Oliver Inc.
W. S. Rockwell Co.
Wallace & Tiernan Inc.

Pumps, Hydrant:
W. S. Darley & Co.
Jos. G. Pollard Co., Inc.

Pumps, Hydraulic Booster:
Ross Valve Mfg. Co.

Pumps, Sewage:
Allis-Chalmers Mfg. Co.
DeLaval Steam Turbine Co.
C. H. Wheeler Mfg. Co.
Worthington Corp.

Pumps, Sump:
DeLaval Steam Turbine Co.
Layne & Bowler Pump Co.
C. H. Wheeler Mfg. Co.
Worthington Corp.

Pumps, Turbine:
DeLaval Steam Turbine Co.
Layne & Bowler, Inc.
Layne & Bowler Pump Co.

Recorders, Gas Density, CO_2, NH_3, SO_2, etc.:
Permutit Co.
Wallace & Tiernan Inc.

Recording Instruments:
Builders-Providence, Inc. (Div., B-I-F Industries, Inc.)
Burgess-Manning Co., Penn Instruments Div.
Infilco Inc.
Minneapolis-Honeywell Regulator Co.
Simplex Valve & Meter Co.
Wallace & Tiernan Inc.

Reservoirs, Steel:
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
R. D. Cole Mfg. Co.
Graver Tank & Mfg. Co.
Hammond Iron Works
Pittsburgh-Des Moines Steel Co.
Sparling Meter Co.

Sand Expansion Gages; see Gages

Sleeves: see Clamps

Sleeves and Valves, Tapping:
James B. Clow & Sons
M & H Valve & Fittings Co.
Mueller Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.

Sludge Blanket Equipment:
General Filter Co.
Graver Water Conditioning Co.
Permutit Co.

Sodium Chloride:
International Salt Co., Inc.

Sodium Fluoride
American Agricultural Chemical Co.

Sodium Hexametaphosphate:
Calgon Co.

Sodium Hypochlorite:
John Wiley Jones Co.
Wallace & Tiernan Inc.

Sodium Silicate:
Philadelphia Quartz Co.

Sodium Silicofluoride
American Agricultural Chemical Co.
Tennessee Corp.

Softeners:
Cochrane Corp.
Dorr-Oliver Inc.
General Filter Co.
Graver Water Conditioning Co.
Hungerford & Terry, Inc.
Infilco Inc.
Permutit Co.
Roberts Filter Mfg. Co.
Walker Process Equipment, Inc.

Softening Chemicals and Compounds:
Calgon Co.
Cochrane Corp.
General Filter Co.
Infilco Inc.
International Salt Co., Inc.
Permutit Co.
Tennessee Corp.

Standpipes, Steel:
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
R. D. Cole Mfg. Co.
Graver Tank & Mfg. Co.

Hammond Iron Works
Pittsburgh-Des Moines Steel Co.

Steel Plate Construction:
Alco Products, Inc.
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
R. D. Cole Mfg. Co.
Graver Tank & Mfg. Co.
Hammond Iron Works
Pittsburgh-Des Moines Steel Co.

Stops, Curb and Corporation:
Hays Mfg. Co.
Mueller Co.

Storage Tanks: see Tanks

Strainers, Suction:
James B. Clow & Sons
M. Greenberg's Sons
R. D. Wood Co.

Surface Wash Equipment:
Cochrane Corp.
Permutit Co.

Swimming Pool Sterilization:
Builders-Providence, Inc. (Div., B-I-F Industries, Inc.)
Omega Machine Co. (Div., B-I-F Industries, Inc.)
Proportioneers, Inc. (Div., B-I-F Industries, Inc.)
Wallace & Tiernan Inc.

Tanks, Steel:
Alco Products, Inc.
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
R. D. Cole Mfg. Co.
Graver Tank & Mfg. Co.
Hammond Iron Works
Pittsburgh-Des Moines Steel Co.

Tapping-Drilling Machines:
Hays Mfg. Co.
Mueller Co.
A. P. Smith Mfg. Co.

Tapping Machines, Corp.:
Hays Mfg. Co.
Mueller Co.

Taste and Odor Removal:
Builders-Providence, Inc. (Div., B-I-F Industries, Inc.)
Cochrane Corp.
General Filter Co.
Graver Water Conditioning Co.
Industrial Chemical Sales Div.
Infilco Inc.
Permutit Co.
Proportioneers, Inc. (Div., B-I-F Industries, Inc.)
Wallace & Tiernan Inc.

Turbidimetric Apparatus (For Turbidity and Sulfate Determinations):
Wallace & Tiernan Inc.

Turbines, Steam:
Allis-Chalmers Mfg. Co.
DeLaval Steam Turbine Co.

Turbines, Water:
Allis-Chalmers Mfg. Co.
DeLaval Steam Turbine Co.

Valve Boxes:
James B. Clow & Sons
Ford Meter Box Co.
M & H Valve & Fittings Co.
Mueller Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
Trinity Valley Iron & Steel Co.
R. D. Wood Co.

Valve-Inserting Machines:
Mueller Co.
A. P. Smith Mfg. Co.

Valves, Altitude:
Golden-Anderson Valve Specialty Co.
W. S. Rockwell Co.
Ross Valve Mfg. Co., Inc.
S. Morgan Smith Co.

Valves, Butterfly, Check, Flap, Foot, Hose, Mud and Plug:
Builders-Providence, Inc. (Div., B-I-F Industries, Inc.)
Chapman Valve Mfg. Co.
James B. Clow & Sons
DeZurik Corp.
M. Greenberg's Sons
Kennedy Valve Mfg. Co.
M & H Valve & Fittings Co.
Mueller Co.
Henry Pratt Co.
Rensselaer Valve Co.
W. S. Rockwell Co.
S. Morgan Smith Co.
R. D. Wood Co.

Valves, Detector Check:
Hersey Mfg. Co.

Valves, Electrically Operated:
Builders-Providence, Inc. (Div., B-I-F Industries, Inc.)
Chapman Valve Mfg. Co.
James B. Clow & Sons
Darling Valve & Mfg. Co.
DeZurik Corp.
Golden-Anderson Valve Specialty Co.
Kennedy Valve Mfg. Co.
M & H Valve & Fittings Co.
Mueller Co.
Henry Pratt Co.
Rensselaer Valve Co.
W. S. Rockwell Co.
A. P. Smith Mfg. Co.
S. Morgan Smith Co.

Valves, Float:
James B. Clow & Sons
Golden-Anderson Valve Specialty Co.
Henry Pratt Co.
W. S. Rockwell Co.
Ross Valve Mfg. Co., Inc.

Valves, Gate:
Chapman Valve Mfg. Co.
James B. Clow & Sons
Darling Valve & Mfg. Co.
DeZurik Corp.
Dresser Mfg. Div.
Kennedy Valve Mfg. Co.
Ludlow Valve Mfg. Co., Inc.
M & H Valve & Fittings Co.
Mueller Co.
Rensselaer Valve Co.
W. S. Rockwell Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Valves, Hydraulically Operated:
Builders-Providence, Inc. (Div., B-I-F Industries, Inc.)
Chapman Valve Mfg. Co.
James B. Clow & Sons
Darling Valve & Mfg. Co.
DeZurik Corp.
Golden-Anderson Valve Specialty Co.
Kennedy Valve Mfg. Co.
F. B. Leopold Co.
M & H Valve & Fittings Co.
Mueller Co.
Henry Pratt Co.
Rensselaer Valve Co.
W. S. Rockwell Co.
A. P. Smith Mfg. Co.
S. Morgan Smith Co.
R. D. Wood Co.

Valves, Large Diameter:
Chapman Valve Mfg. Co.
James B. Clow & Sons
Darling Valve & Mfg. Co.
Golden-Anderson Valve Specialty Co.
Kennedy Valve Mfg. Co.
Ludlow Valve Mfg. Co., Inc.
M & H Valve & Fittings Co.
Mueller Co.
Henry Pratt Co.
Rensselaer Valve Co.
W. S. Rockwell Co.
A. P. Smith Mfg. Co.
S. Morgan Smith Co.
R. D. Wood Co.

Valves, Regulating:
DeZurik Corp.
Foster Eng. Co.
Golden-Anderson Valve Specialty Co.
Minneapolis-Honeywell Regulator Co.
Mueller Co.
Henry Pratt Co.
W. S. Rockwell Co.
Ross Valve Mfg. Co.
S. Morgan Smith Co.

Valves, Swing Check:
Chapman Valve Mfg. Co.
James B. Clow & Sons
Darling Valve & Mfg. Co.
Golden-Anderson Valve Specialty Co.
M. Greenberg's Sons
M & H Valve & Fittings Co.
Mueller Co.
Rensselaer Valve Co.
W. S. Rockwell Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Venturi Tubes:
Builders-Providence, Inc. (Div., B-I-F Industries, Inc.)
Burgess-Manning Co., Penn Instruments Div.
Infilco Inc.
Simplex Valve & Meter Co.

Waterproofing:
Barrett Div.
Inertol Co., Inc.
Koppers Co., Inc.

Water Softening Plants; see Softeners

Water Supply Contractors:
Layne & Bowler, Inc.

Water Testing Apparatus:
W. A. Taylor & Co.
Wallace & Tiernan Inc.

Water Treatment Plants:
American Well Works
Chain Belt Co.
Chicago Bridge & Iron Co.
Cochrane Corp.
Dorr-Oliver Inc.
Etablissements Degremont
General Filter Co.
Graver Water Conditioning Co.
Hammond Iron Works
Hungerford & Terry, Inc.
Infilco Inc.
Permutit Co.
Pittsburgh-Des Moines Steel Co.
Roberts Filter Mfg. Co.
Walker Process Equipment, Inc.
Wallace & Tiernan Inc.

Well Drilling Contractors:
Layne & Bowler, Inc.

Wrenches, Ratchet:
Dresser Mfg. Div.

Zeolite: see Ion Exchange Materials

A complete Buyers' Guide to all water works products and services offered by AWWA Associate Members appears in the 1957 AWWA Directory.